Validation Isotach model by means of "Barendrechtse weg"

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Abstract

The report "Validation settlement Barendrechtse weg" describes the validation of the Isotach model by means of the measurement of settlement during the construction of a road near the "Barendrechtse weg". The Isotach model predicts the settlement of a structure, which is founded on soft soil; the settlement is caused by consolidation and creep. The Isotach model needs parameters that can be easily determined in the laboratory. The validation of the model increases it applicability in civil engineering.
Recent work in this project for the Waardse Alliantie has enhanced and improved the Isotach model. The model used an artificial sill stress, which made interpretation ambiguous. The sill stress has been abandoned. Further improvements were reached in the determination of the preconsolidation stress and the strain dependent permeability. Analysis of the data as obtained for the "Waardse Alliantie" and for the "Barendrechtse weg" reveals the success of those improvements.

Using the parameters, as determined by the K_0-CRS tests and using a strain dependent permeability, the calculated settlement is very close to the measurements even if factors of uncertainty in load and parameters of the soil exist. Variations in the shape and height of the external load introduce an uncertainty of 0.1 m. Variations in the water content of the load introduce another 0.1 m of uncertainty.

The pore pressures measured in a layer are higher than predicted. This difference points to an in situ permeability that is lower than measured in the laboratory. A similar result: in situ permeability is lower than determined in the laboratory, was found when analysing the pore pressure data of the "Waardse Alliantie". However, since the pore pressures of only one layer was analysed, it is not possible to conclude that the permeability of all the layers was over-estimated by the laboratory tests.

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Executive Summary

Significance. Roads and railroads are traditionally built using local material, such as sand and clay. The foundation of a road, particularly in the Western and Northern part of the Netherlands, consists of soft and even very soft soils. The low permeability of the soil delays the construction of a road, since it hampers consolidation. Next to consolidation soft soils are susceptible to creep. Both consolidation and creep cause settlement of the road, which leads to damage. Since the process of raising a road above the ground level takes time, the main part of the consolidation occurs during the construction process of the road, before it fulfils its purpose. Later, when the road is opened for traffic, creep dominates the deformations. These deformations cause mainly financial damage. The organisation that has to maintain the quality of the road has to invest in repair actions. Society looses money due to prolonged periods of additional traffic jams, when the road or part of it, is blocked for use.

Modelling. The usual techniques to predict the settlement of a road deal with consolidation and creep in a separate way. The Isotach model deals with them in an integrated way. Deformation of the soil causes excess pore pressures, which will have to dissipate. Simple hydraulic equations using the permeability of the layers of soil deal with the resulting ground water flow. Parameters describing the behaviour of the soil skeleton under compression determine its deformation. Hydraulics and the soil mechanics interact: deformation of the soil skeleton changes the permeability and thus the ground water flow. Assuming that the pore water is hardly compressible, the mass balance of the water and the deformation of the soil skeleton determine the exact value of the settlement.

In the previous paragraph it is explained which parameters determine the settlement: the mechanical behaviour of the soil skeleton and the hydraulic permeability of the soil. These parameters can be measured in a K_0-CRS test, in which a sample of soft soil is loaded with a constant rate of strain. The parameter responsible for creep is determined by introducing either a stage with constant stress or constant strain.

Purpose. The Isotach model predicts settlement, based on laboratory tests, without the need for extrapolation of measured settlement data, obtained when the road is under construction. The question is: "What is the prediction worth?" In order to ascertain the correctness of the prediction, the Isotach model has to be validated: compared with experimental results. Differences between prediction and measurement reveal the correctness of the model. In stead of predictions one can use postdictions for the purpose of validation as well, as long as one can withstand the temptation to improve the model or its parameters till the point that there is an excellent agreement between measurement and calculation.

Background. Earlier validation of the model has revealed differences. The sill stress, which has been introduced to quench numerical oscillations, has shown to be not a sound method. By varying the sill stress 'virtually any settlement' could be calculated. The origin of the numerical oscillations is firmly based in the Isotach model itself. If a stress is added to a layer without any stress, the resulting deformation is infinite. A physically sound solution for this stalemate is presented by den Haan in 2000. Unfortunately the sector lacks interest in its implementation, mainly because of the complicated nature and the high costs involved. The sill stress has been replaced by a limit stress in order to limit its influence.

Another improvement that has been introduced during the work for the "Waardse Alliantie" in the implementation of the Isotach model is the strain dependent permeability. Due to the deformation of the soil, the pores become narrower. So when strain increases, permeability decreases.

The validation of the settlement as measured during the construction of the road near the "Barendrechtse weg", will show whether the concept of strain dependent permeability is an accidental or a structural improvement.

Scope. In situ measurements were performed by both GeoDelft and the contractor that has built the road. Both the settlement and the geometry of the embankment were measured. These measurements are partially independent. The contractor has used settlement plates; GeoDelft has used a settlement hose, which gives information in a section line. The settlement as determined by both companies, is in good agreement with each other: they differ less than 5%. The differences in the outer geometry of the road, as determined by GeoDelft and by the contractor, are larger: 15%.

In two section lines samples were taken. K_0-CRS tests and oedometer tests have been performed on the samples from section line "raai B", early 2000. These tests determine the parameters for the Isotach model. Additionally, now that strain dependent permeability can be accounted for, the K_0 tests have been re-evaluated in order to determine the strain dependent permeability.

Calculation of the settlement was performed by the MSettle suite, version 6.7. The determination of the strain dependent permeability is still in its development stage: a spreadsheet. The a and b parameter of the Isotach model have been calculated manually and this calculation is checked by Compress, version 1.0. Differences between manual and automatic calculation are small, of the order of 10%.

Results. The calculated settlement is very close to the measurements, even if uncertainty in the height and shape of the external load and its water content exist. Pore pressure measurements show that the permeability in one of the layers was over-estimated by the laboratory tests. A similar result was found for the "Waardse Alliantie" project.

Conclusions. The calculations performed for the "Barendrechtse weg" show that the agreement between calculated and measured settlements is very good. This result is obtained using the mechanical and hydraulical parameters of the sub soil, as determined in the laboratory. As a result it should be concluded that the combination of the Isotach model and the K0-CRS tests is a sound method to predict settlement due to the construction of an embankment.

The concept of the strain dependent permeability, as introduced in the "Waardse Alliantie" project, is also a sound concept.

The pore pressures measured in one layer are higher than predicted. If the permeability is reduced to 30% of the permeability measured in laboratory, the agreement for pore pressure becomes better but, at the same time, the agreement for settlement becomes less good. The factor 3 is probably not applicable for all the layers but, as only one transducer has relevant pore pressure measurements, the evaluation of the correct factor for each layer is not possible.

Recommendations. The demand for "high precision" predictions of settlement can be met if the external load is similarly known in detail. This extends to height and shape of the external load and its geotechnical properties. Deviations of the specifications of the geometry of the load and / or its geotechnical properties should be recorded and passed to the consultant of the contractor as soon as possible. A close monitoring scheme will facilitate the determination of possible deviations from the specifications or from the predicted settlement. Such information will allow for adaptation of the design, if the predicted deviations from the design criteria tend to be too high.

Since uncertainty in the geotechnical parameters of the sub soil will be present, its influence on the predicted settlement has to be accounted for as well. Since the influence of the latter uncertainty is unknown, it should be investigated. Such an analysis will lead to the information which parameter of the Isotach model has a dominant effect on the predicted settlement. If the influence of this parameter leads to an unacceptable uncertainty in the calculated settlement, the natural variation of the according parameter in the soil has to be determined, in order to determine the relevance of a more accurate characterisation of the sub soil.

An integrated approach of settlement and pore pressures leads to a more accurate prediction of the settlement. If high precision prediction of settlement is necessary, pore pressures should be measured as well. They will enable the 'fine tuning' of the parameters during the actual construction process.

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Applicability for the sector

General. The Isotach model for the prediction of settlement has been implemented in the MSettle suite. Analysis of the "Waardse Alliantie" data has shown that the sill stress is not a particularly good parameter, since it influences the calculated settlement in an unacceptable way. Furthermore it was shown that the influence of strain on the permeability of the soil is another parameter which to be accounted for.

Conclusions. The calculations for the validation by means of the "Barendrechtse weg" have shown that the geometry of the external load, i.e. the height and shape of the load and its geotechnical parameters, has to be known in detail if an accurate prediction of the settlement is needed.

The K_0-CRS test is shown to be a good test to determine the parameters that control the settlement.

Limitations. The Isotach model has been tested especially on soft soils. Application of the Isotach model on silty or clayey sands lacks experience. A consultant should be aware of this lack of experience on less compressible soils, if he encounters that type of soils.

Availability. The calculations (postdictions) for settlement and pore pressures were done by means of the MSettle suite, version 6.7. The re-analysis of the K_0-CRS tests was performed by MCompress, version 1.0. The accessibility and user-friendliness of the software ensures a potentially broad distribution of the Isotach model in the geotechnical world, mainly consultancy firms.

Distribution. The Isotach model is being used by a limited group of companies. Information on which specific users is currently not available.

Costs. Application of the Isotach model and K_0-CRS tests involves costs, which are comparable to oedometer tests and e.g. a classic Koppejan calculation.

Risk of failure. Failure of the introduction of the Isotach model in settlement calculation depends on the availability of reliable data. The Isotach model will be applied in those situations where an accurate prediction of settlement is necessary. If the loading steps are not known in detail, the calculation may provide results that differ from the observed settlement. These differences will be attributed to the Isotach model, while the real origin of the differences is mainly caused by the lack of information about loading steps and materials used.

Time span. The Isotach model is available for the civil engineering society. DC partners have put effort in the introduction of the method. The K_0-CRS test, a specific method of laboratory testing, is not yet broadly integrated in the design process in the engineering companies. This will take its time. The method is likely to be introduced by Rijkswaterstaat, which can provide a technology push. Within a few years, maybe five years, it will be more broadly used, especially in those cases where settlements are expected to be high, or when several stages of loading and unloading will take place. Replacement of e.g. classical methods like "Koppejan" will take a decade.

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Societal Relevance of the research

Growing demand for mobility stresses the current infrastructure to the limits, especially in areas with a high economic productivity. Extending the existing infrastructure is the major concern of the current government. Limited financial resources and the growing liberalisation, aggravated by the increasing influence of Brussels, has driven the construction of roads and railroads towards privately owned or controlled parties. Receding resources for the extension of the infrastructure have lead to solutions in the realm of Design, Construct and Maintain. Contractors are free to choose their own solution, as long as a number of vital demands, imposed by the government, is fulfilled.

When responsibility is relocated at the side of the contractor, he will require, next to the costs for the construction process, also an accurate estimate of the future costs. Maintenance caused by settlement of the road is one of the items high on his list. Since construction and maintenance is handled by one party, an integrated optimisation is possible. The impact of this method for construction and maintenance is that the sum of the individual costs can be minimised. A further advantage is that a construction will be ready in a shorter period of time, i.e. the society can harvest the benefits of the investment as soon as possible. However, it should be noted that the bureaucratic procedures substantially prolong the time between the conceptual design and the actual construction of a road.

Design and maintenance has a disadvantage as well. Roads have to be maintained for many decades. In the meanwhile a contractor can go bankrupt, leaving the government with the costs for the maintenance.

This report describes a method for the prediction of the long term settlement of a road or railroad. It enables the user to predict settlement, especially caused by creep in soft soils. These soils are mainly found in the Western part of the Netherlands, in the same area where economic activity is high. The results of this report will be used mainly by civil engineering companies or divisions that will advise a contractor on topics of settlement and deformation.

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1 Introduction

1.1 General

When an embankment is constructed on top of an undisturbed ground level, especially when the subsoil is very soft, a substantial settlement is to be expected. The settlement is not instantaneous. The reason is simple: the soil skeleton is compressible, whereas the pore water is hardly compressible. Due to the compression of the soil skeleton water in the pore structure is squeezed out. Soft soils are usually rather impermeable. The low permeability of the soil restricts the flow of the pore water. As a result the deformation is not instantaneous, but it takes quite some time. From a geotechnical point of view two processes play a role: consolidation and creep. Especially creep is responsible for the long term settlement. The period in which creep is active, coincides with the period a road is to be maintained and serviced. Consolidation is mainly a dominant process during the initial phases of the construction.

For consolidation as well as creep several conceptual models have been developed, which predict settlement. Costs for the management and maintenance of roads and railroads are increasingly important, not in the least while private parties are increasingly kept responsible for the maintenance of roads. This aspect raised the demand for accurate predictions of settlements.

Since there is a market driven demand for a more accurate prediction of settlement than there used to be, a Delft Cluster project was initiated: "Samengestelde constructies". The aim of the project is to arrive at an integrated approach of soil and structure. The emphasis in this project is directed mainly towards the prediction of settlement caused by creep.

Part of the project is the validation of the so-called Isotach model or 'abc model'. This model is described by [den Haan 1994]. An implementation of this model can be found in the MSettle suite, which makes the Isotach model accessible to the geotechnical community. In order to determine how accurately the model can predict long term settlement, calculations using the Isotach model are to be compared to settlements as measured in the field. This is the validation of the Isotach or 'abc model' and of the method to measure its input parameters in the laboratory.

1.2 Barendrechtse weg

Due to the 'Betuwe route' project a new road is to be constructed near the Barendrechtse weg. Since there are cables and pipes in the vicinity of the new road, special emphasis on the accurate predictions of settlements were made. In order to minimise the deformations in the soil, part of the embankment was to be made of EPS, a very light (foamy) material. A close monitoring scheme was set up to serve as an early warning system for the possible damage to the infrastructure. During the construction process the monitoring data have proven to be of great value for the contractor. It was shown that the actual deformations were less than expected. In stead of the rather costly EPS a conventional embankment consisting of sand could be realised. This was a drawback for the purpose that the Barendrechtse weg was chosen: an embankment consisting of light material. Since a lot of data was gathered, the settlement, as measured at the Barendrechtse weg, nevertheless is a good candidate to validate the Isotach model.

Next to the measurement of the in situ settlement, a core boring was made. Samples were collected from the core. On these samples K_0 tests and oedometer tests have been performed, in order to determine the 'abc' parameters. Results of these tests have been reported in [den Adel 2002]. In this report part of the settlement data (2000) was compared with postdictions based on the Isotach model.

In this report all settlement data, as available in October 2002, were compared with the postdictions as made by the Isotach model. The model itself was improved as well as our understanding of the parameters to be used in the model during the work on the 'Samengestelde constructies' project. Furthermore monitoring data of pore pressures were added to the analysis and validation. The latter data have proven to be of importance.

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2 Recent developments

The current report is a follow up of report [den Adel 2002]. In the nine months between the latter report and this one developments on the implementation of the Isotach model have taken place. These developments have been originated by the work that has been done for the "Waardse Alliantie" in the project "Samengestelde constructies". The developments comprise the concept of limit stress, pre overburden pressure and the concept of strain dependent permeability.

2.1 Stresses

Part of the project "Samengestelde constructies" was the prediction of settlement in two sections of the Betuwe railroad [den Haan 2002] under construction. While comparing the actual settlement data with the postdicted values, deviations were detected. The main reason for these deviations was shown to be the sill stress.

The sill stress is an artificial stress, believed to have been present on the ground level. As pointed out in [Sellmeijer 2002] one of the artefacts of the Isotach model is an anomaly for layers of soil, which do not have any stress at all. In such layers the Isotach model predicts an infinite strain, when the stress is incremented from zero to non zero. Fortunately in nature layers with zero stress are infinitely thin, so no real harm is to be expected. However, in the numerical implementation of the Isotach model no such thing as infinitely thin layers exist, so a real problem is created by the numerical schematization.

Even when strain is relatively low, the Isotach model calculates a large strain. Such a large strain causes a huge excess pore water pressure, which has to dissipate. As a result pore pressures in adjacent layers start to grow as well. Due to the extra pore pressures in adjacent layers, deformation is hampered. In this way adjacent layers interact, influencing the mutual pore pressures. This leads to an unstable solution of the mathematical model: pressures and strains start to oscillate.

The origin of this instability is the relation between strain and stress:

$$\frac{1}{\hat{p}_{\mathrm{j}}^{a}}\frac{\mathrm{d}}{\mathrm{d}t}\left(\frac{\hat{v}_{\mathrm{j}}}{v_0}\hat{p}_{\mathrm{j}}^{a}\right) = -\varepsilon_0\left(\frac{\hat{v}_{\mathrm{j}}}{v_0}\right)^{\frac{1}{c}+1}\left(\frac{\hat{p}_{\mathrm{j}}}{p_0}\right)^{\frac{b}{c}} \tag{2.1}$$

If p_0 is zero or nearly zero, the right hand side of the equation becomes infinite respectively very large. In order to overcome the infinite strain anomaly originally a sill stress was introduced. In stead of the value for p_0 a slightly larger value was used: p_0+p_{sill}. Virtually a small extra load was added on the ground level, before calculations had started. This quenches the numerical instability related to the infinite right hand side of the equation. One of the side effects is that the sill stress influences the predicted settlement, which may lead to rather large deviations between settlement curves at different sill stresses.

During the analysis of the data [den Haan 2002] it was shown that the sill stress was a parameter which dominated the calculation of settlement. By varying the sill stress, 'virtually any settlement' could be calculated. Results may deviate between 10% and 50% with respect to the observed settlement.

The physically sound way out of this anomaly is to assume that very low stresses hardly yield any strain, contrary to what the Isotach model predicts. A conceptual model for this assumption has been presented in the Noordwijkerhout paper [den Haan 2000]. This model concerns the secular strain. An ultimate strain has been defined, ε_u:

$$\varepsilon_m - \varepsilon = \frac{\ln\left(\frac{\sigma_{vm}'}{\sigma_v'}\right)}{\left(\frac{1}{b} + \frac{\ln\left(\frac{\sigma_{vm}'}{\sigma_v'}\right)}{\varepsilon_m - \varepsilon_u}\right)} \tag{2.2}$$

The equation is visualized in Figure 2.1.

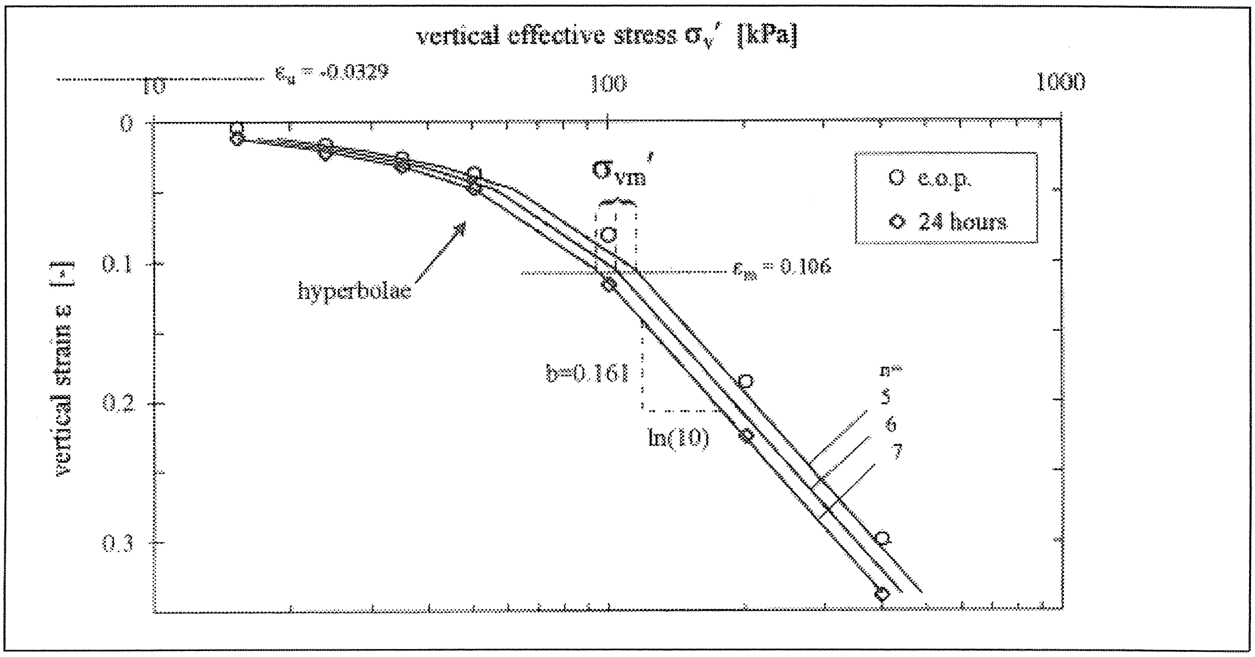

Figure 2.1 *Hyperbolic topping at low stresses according to [den Haan 2000]*

The implementation of this idea is unfortunately very complicated and therefore costly; it is not an item with a high priority for the participating market parties.

A mathematical way out, analogous to the Noordwijkerhout paper, is to introduce a limit stress, limiting the infinite strain contribution at low stresses. In the calculations it is assumed that the stress p_0 in a layer has a lower limit: the limit stress. The limit stress has a similar effect in equation (2.1) as the sill stress. Yet the two differ in influence. The limit stress can be defined for each layer separately. It is acting in one or two layers, when the effective stress in these layers is lower than the limit stress. On the contrary, the sill stress is constant for all layers and it acts in all layers, whether necessary for the quenching of oscillations or not.

The pre overburden pressure –POP for short- is assumed to be a constant value in each layer. The POP is defined as:

$$POP = p_g - \sigma'_v \tag{2.3}$$

p_g is the preconsolidation stress. It relates to the stress a soil sample has been subjected to in the past. The value of p_g is derived from a K_0-CRS or compression test. Determination of its value is explained in 3.5.2. σ'_v is the effective stress in the sample. The effective stress in the sample, when it was still in the terrain, is estimated from the volumetric weight and thickness of the layers above it.

The value p_0 in equation (2.1) has been replaced by:

$$p_0 = MAX\left(\left(\sigma'_v + POP\right), p_{\text{limit}}\right) \tag{2.4}$$

assuming an over consolidation ratio of 1. In equation (2.4) POP must be considered as a constant, a representative value for a layer. Its value is an experimental result, as derived for a sample in that layer. The value of σ'_v varies with depth in a layer.

This approach as in equation (2.4) is shown [den Haan 2002] to be a solution that circumvents the infinite strain anomaly. Moreover it leads to a much better agreement between measured settlements and their postdiction than previously could be obtained.

2.2 Strain dependent permeability

The Isotach model is specifically appropriate to be used when settlements are large. The classical Koppejan model uses linear strain. This leads to the theoretical possibility of a strain being 100%. The Isotach model uses natural strain, which is a logarithmic flavor of linear strain. How large stresses may ever be, a strain of 100% can never be reached.
The Isotach model is likely to be applied when settlements - and thus strains - are expected to be high. When strains are high, the pore skeleton is deformed rather rigorously. The deformations have a large influence on the permeability of the soil. In K_0 tests the influence of a decreasing permeability can be clearly seen by the raise in pore pressure in the sample at constant rate of strain, see Figure 2.2, since pore pressure is inversely proportional to the permeability of the soil, according to equation:

$$k(e) = \frac{1}{2}\frac{Vh\gamma_w}{u_b} \tag{2.5}$$

where k(e) is the permeability at void ratio e, V is the actual speed, h is the actual height of the sample, γ_w is the water weight and u_b is the pore pressure at the bottom of the sample.

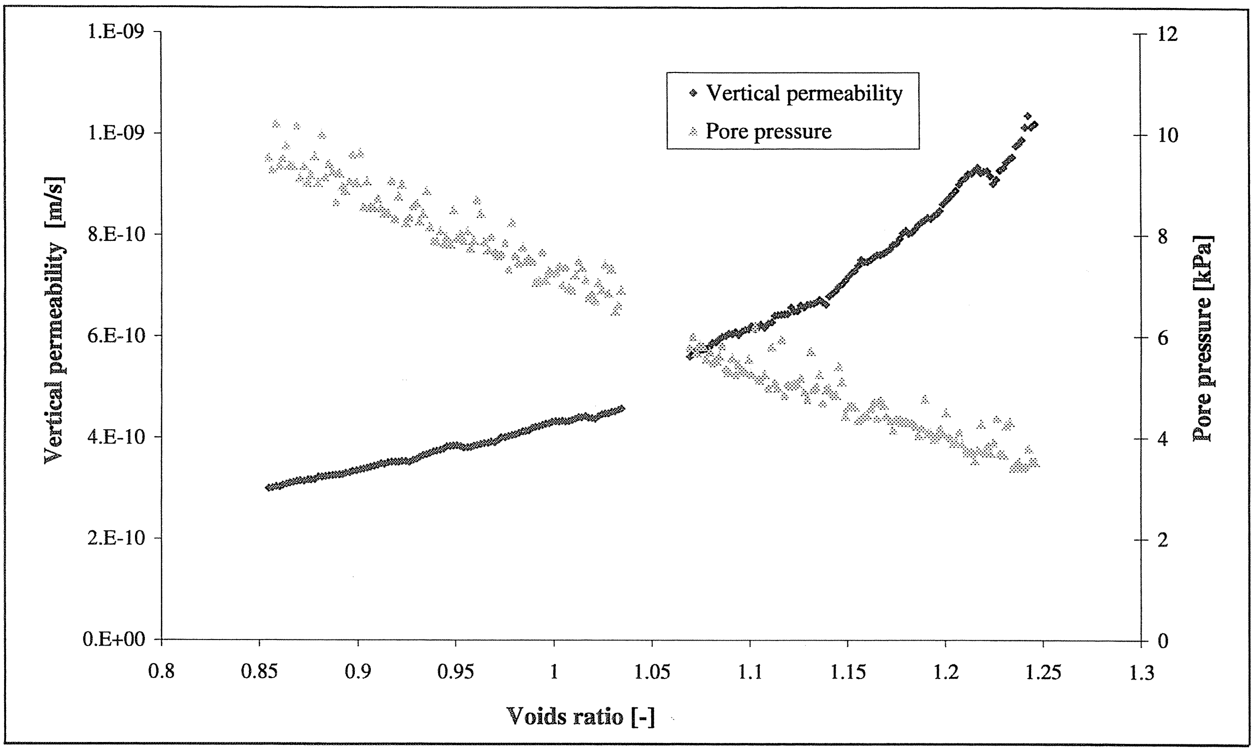

Figure 2.2 *Evolution of pore pressure and permeability as a function of voids ratio at a constant rate of strain*

In calculations reported in [den Adel 2002], permeability was kept constant. In the "Waardse Alliantie" report [den Haan 2002] it was shown that the assumption of a constant permeability is not good agreement with the measured settlement. In the MSettle suite a variable permeability was implemented:

$$k(e) = k(e_0) \times 10^{\frac{e-e_0}{C_{ke}}} = k(e_0) \times 10^{\frac{\varepsilon}{C_{k\varepsilon}}} \quad \textbf{(2.6)}$$

where e is the actual void ratio, e_0 is the initial void ratio, $k(e_0)$ is the permeability at the initial void ratio and k(e) is the permeability at void ratio e. The value of C_{ke} is determined experimentally in a K_0 test.

3 Experimental data

3.1 Geometry

The geometry consists of two parts: the soil, i.e. the foundation for the road and the road itself.

3.1.1 Soil

The subsoil was characterized by a set of layers of material. Since in each of the two sections lines (raai A and B) just one boring is made, in a section line the interface between adjacent layers is supposed to be horizontal.

Material	depth from [GL - m]	depth till [GL -m]
Dunkirque clay	0.00	-0.77
Dunkirque sand	-0.77	-1.50
Dunkirque sand	-1.50	-2.00
Holland peat	-2.00	-3.11
Gorkum heavy 1	-3.11	-5.63
Holland peat	-5.63	-7.60
Gorkum light	-7.60	-8.98
Gorkum heavy 2	-8.98	-9.52
Gorkum light	-9.52	-10.00
Gorkum heavy 2	-10.00	-11.25
Gorkum heavy 1	-11.25	-12.00
Base peat	-12.00	-12.40
Pleistocene sand	-12.40	

Table 3.1 *The layers in the soil underneath the new road (Raai B)*

3.1.2 Road

From a civil engineering point of view a road at a height of several meters above ground level had to be constructed. The road was constructed in several steps, spaced in time. The first steps applied dredging techniques. The later steps consisted of rather conventional techniques: dumping of sand. In order to minimize the time needed for a safe construction of the road, vertical drainage of the soil was applied. The distance between the drains is 1 m; the drains are positioned in a triangular pattern.

In the first step two embankments of roughly 1 m high were constructed at both sides of the new road. Between these two lateral embankments a mixture of water and sand was pumped. As a prelude to the second step, after waiting for a while in order to drain excessive water, the embankments at both sides were heightened again. Thereafter an additional mixture of sand and water was pumped between the two lateral embankments.

The road was now roughly two meters high. Since applying dredging techniques for further heightening of the road would mean a serious hazard for the adjacent houses, the last steps were applied using the conventional technique of earth moving lorries and bulldozers. The geometry of the successive steps of the road was measured by the contractor as well as by GeoDelft. The contractors geometry is shown in Figure 3.1. Note that the zakbaak has been renamed for simplicity sake into A5, whereas the contractor uses A7_5.

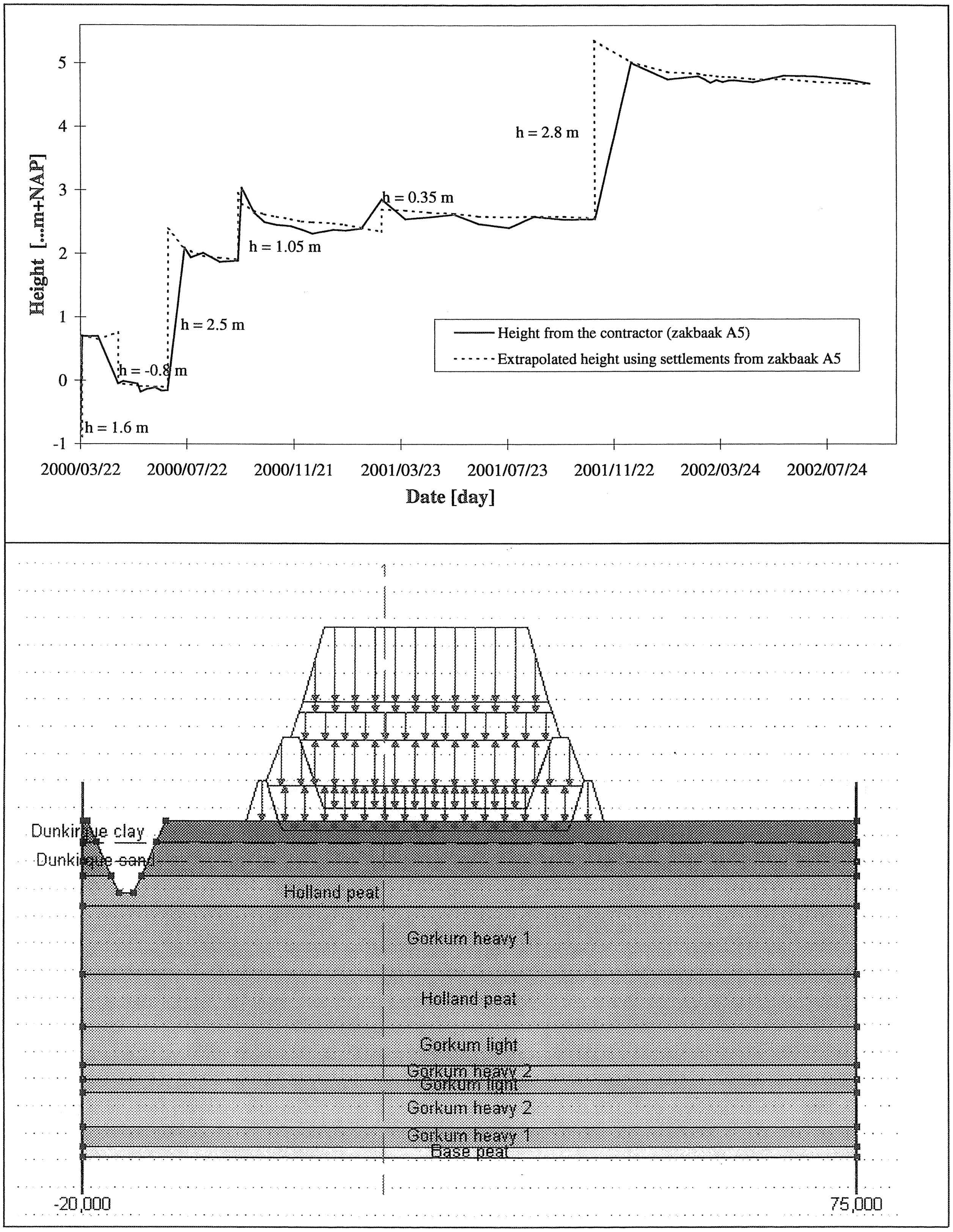

Figure 3.1 *The geometry in steps, as determined by the contractor*

The geometry as determined by GeoDelft is shown in Figure 3.2.

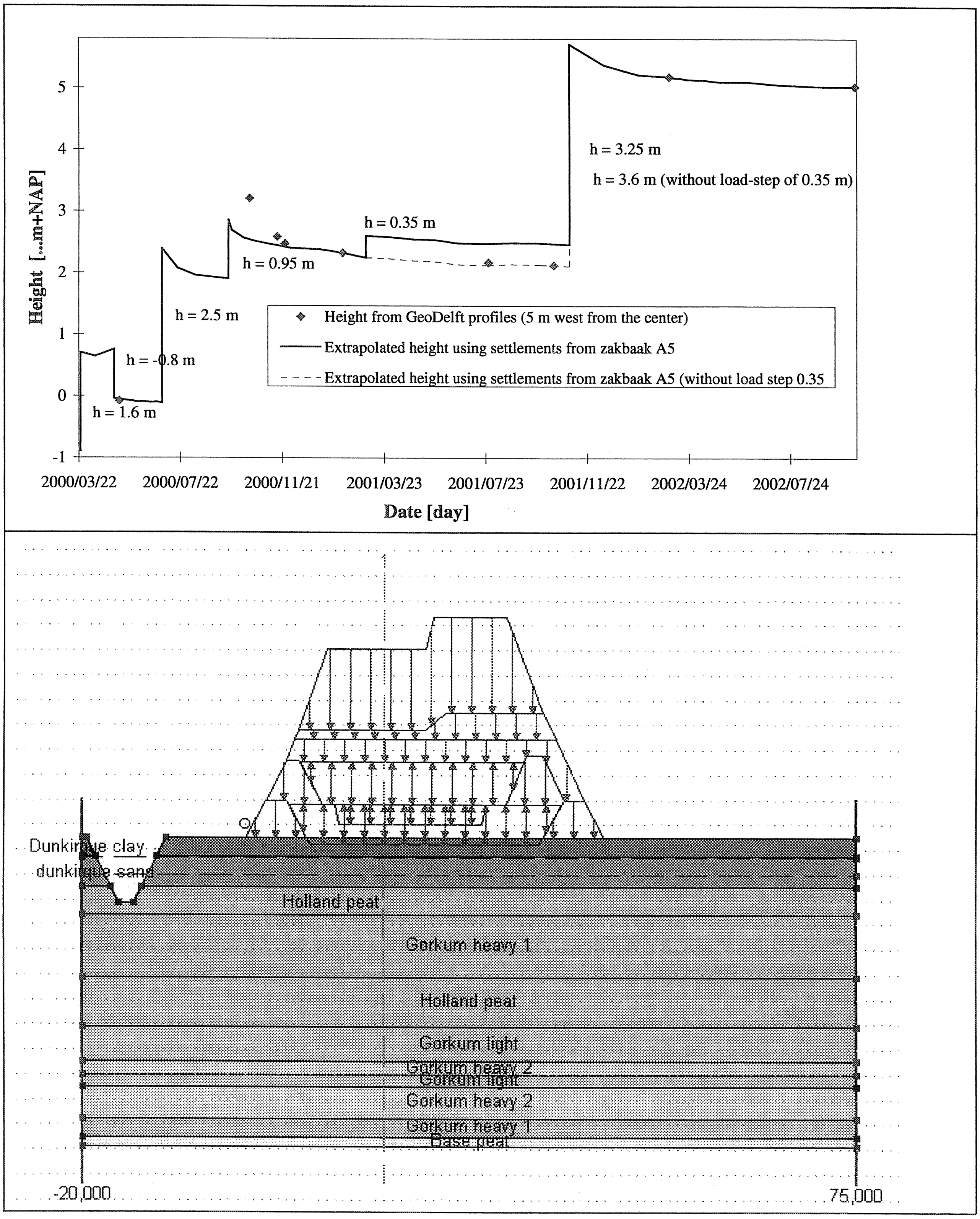

Figure 3.2 *The geometry in steps, by GeoDelft*

The geometry as determined by the contractor is in a section line, which is 5 m to the north of section line "raai B", see Annex A. Note that the geometry determined by GeoDelft lacks information between April 2000 and October 2000. To fill in the missing data, the contractors geometry has been inserted during that period.

The building steps, as used in the calculations, are explained in Table 3.2:

Day	Date	Action	γ [kN/m^3]	Height [m]		Remarks
				From	Till	
0	2000-03-11	Excavate	-19.2	-0.5	-0.8	Build lateral embankments
12	2000-03-23	Heighten	20	-0.8	+0.8	Add sand water mixture
26	2000-04-06	Excavate	-20	+0.8	-0.8	Remove wet sand
26	2000-04-06	Heighten	18	-0.8	+0.8	Sand is partially dry
44	2000-04-24	Excavate	-18	+0.8	0	Heighten lateral embankments
117	2000-07-06	Heighten	20	0.0	2.5	Add sand water mixture
131	2000-07-20	Excavate	-20	2.5	0	Remove wet sand
131	2000-07-20	Heighten	18	0	2.5	Sand is partially dry
192	2000-09-19	Heighten	18	2.5	3.55	Heighten with dry sand
332	2001-02-06	Heighten	18	3.55	3.90	Heighten with dry sand
599	2001-10-31	Heighten	18	3.90	6.70	Heighten with dry sand

Table 3.2 *The steps in the construction of the new road*

It should be noted that this table differs form the corresponding table in report [den Adel 2002]. Notes of the contractor on the geometry suggest that the sand water mixtûre was applied on day 12, in stead of day 25. Two weeks later the water in the mixture is supposed to have drained. As a result the sand was supposed to have been drained on day 26, in stead of day 39.

The excavations and heightenings on day 26 and day 131 are virtual actions, as needed in the calculations to simulate the fact that within two weeks after applying the sand water mixture, the water has drained. In this period of drainage it is assumed that the volumetric weight of the sand reduces from 20 to 18 kN/m^3.

The step at day 332 (2001-03-01) as in the data of the contractor, (see Figure 3.3) is not visible in the data of GeoDelft. No measurements directly before and after the step are available. By comparing data of the contractor and the data of GeoDelft it can not be determined whether the height of 4.55m is real or a flawed measurement. Nevertheless there is reason to believe that a step has been added. It is found in Figure 3.3 that the height of the sand gradually increases during the year 2001, from 4.2m to 4.55m. If the data point at day 332 is an error, for what reason the height of the sand increases of its own by 35 cm within a year?

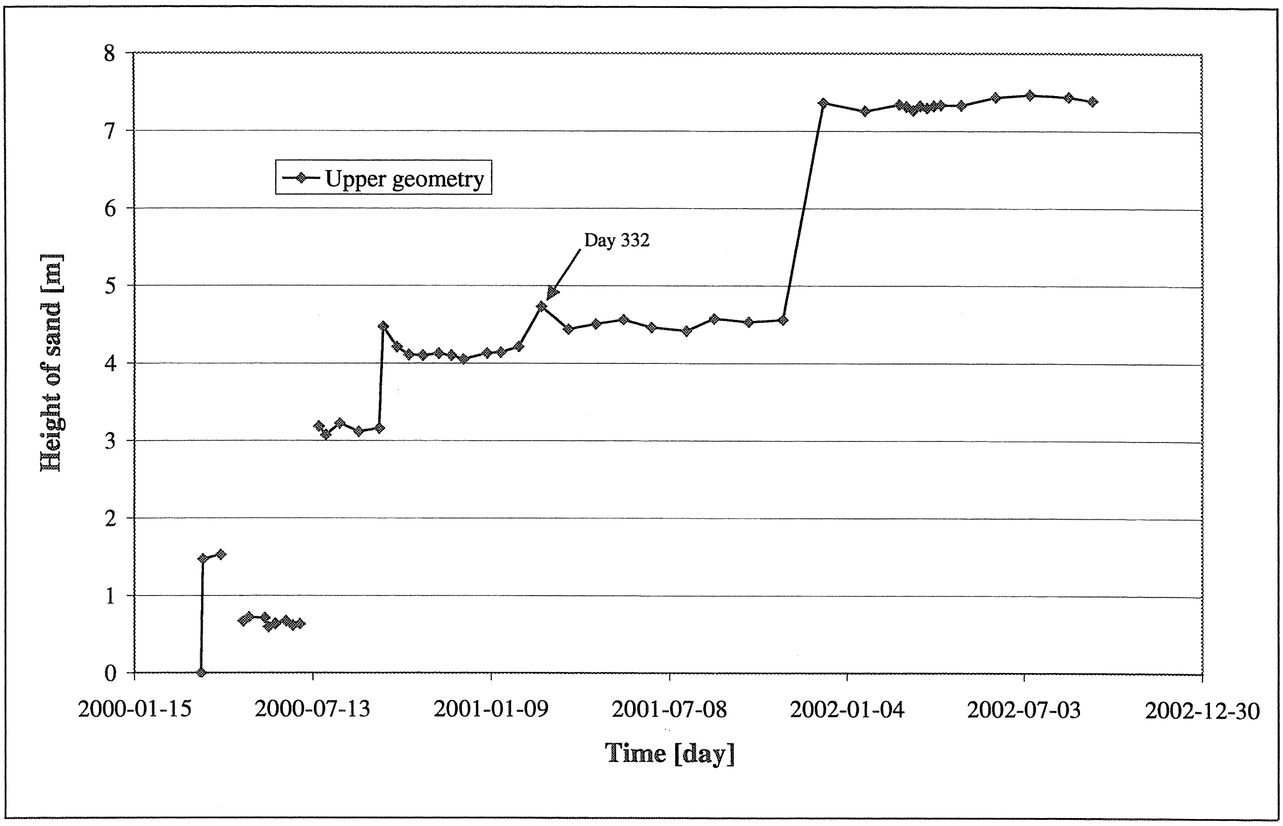

Figure 3.3 The height of the sand, corrected for settlement

Note that in Figure 3.2 the effect of settlement is visible in the height of the sand, whereas in Figure 3.3 the height has been corrected for the actual settlement.

3.1.3 Vertical drainage

There are reasons to believe that the dates on which the vertical drainage was placed, as used in report [den Adel 2002], are not correct. Interviewing people that have visited the building site yields the following deviations regarding the time vertical drainage was placed.

- Initially the vertical drainage was applied underneath the two embankments at the side of the road under construction. This coincides with day 0: 2000-03-11, when the excavation of the top layer took place. The lateral embankments were made by excavating the central section between the lateral embankments.
- Later, after the excavation but before the sand water mixture was added on day 12, the vertical drainage of the middle section was placed.

Recollection in the human mind of the dates from two years ago may be flawed. However, the following argument enhances our belief that the recollected dates are correct. Since the standard equipment for vertical drainage is not capable of penetrating more than half a meter of sand, vertical drainage has to be placed before the construction of the lateral embankments, which are 1.5 m high. Furthermore it has to be done before the mixture of sand and water is fed between the embankments, since this action makes the central section inaccessible for heavy terrain equipment.

In report [den Adel 2002] the following paragraph is listed:
"Drainage is supposed to be placed on date 51. From notes by the contractor it is not clear on which date drainage has been placed: 39 or 51 days after excavation of the top layer."
The paragraph lists a difference of 12 days. This difference in time coincides with the difference in time between the actions on day 0 and 12. This coincidence may be accidental as well as non-

accidental, in the latter case confirming the new dates of placing vertical drainage. A period of 12 days is the ultimate time: just before the sand water mixture was added. Later than day 12 is not possible. It is even possible that vertical drainage in the central section was already present at day 0.

The drains have been installed in a triangular pattern. Their distance is 1 m. This value has been confirmed by two independent sources. Information provided by the contractor gives a clear statement: 1m. The former project leader, visiting the site when vertical drainage was placed, uses deduction to arrive at 1m as well, by excluding 0.5m and 2m distance. The values 0.5, 1 and 2m are standardized distances in vertical drainage.

This means that the soil under the lateral embankments was always drained; however, the soil between the lateral embankments might not be drained during excavation. It was drained during the first step. Since the embankments are far away from the center of the road, excavation is considered to be not drained.

3.2 Settlement

Two techniques were applied to determine the settlement:

- Settlement plates
- Settlement hoses

3.2.1 Settlement plate

A settlement plate is a plate roughly 0.5m by 0.5m, which rests on the initial ground level. Perpendicular to this plate a rod is welded. The height of the top of the rod is measured using surveying techniques. Roughly a dozen of settlement plates were installed by the contractor, see Annex A. For the section line, in which the soil and its characteristic parameters are schematized, settlement plates A7_4, A7_5 and A7_6 are appropriate (A4, A5 and A6 in Annex A).

3.2.2 Settlement hose

The disadvantage of a settlement plate is that it produces information on a single point only. A settlement hose overcomes this disadvantage by allowing for measurements on a line. A hose is buried under the road under construction, perpendicular to the axis of the road. Both ends of the hose extend freely in the air. The hose is filled with water. A pore pressure transducer will be pulled through the hose. Since the water in the hose is under atmospheric pressure, the pressure as measured by the pore pressure transducer in the hose, is an indicator for the vertical distance between the transducer and the water level in the hose. When the height of the water level in the hose with respect to a datum, e.g. NAP, is accurately measured, the depth of the pore pressure transducer can be derived from the measured pore pressures and the height of the water level. Two settlement hoses were buried, before the construction of the road started, in two section lines, "raai A" and "raai B". The settlement hose is only useful to determine deformation, since it is virtually impossible to bury the hose completely horizontal. Before the heightening steps will start, a zero measurement determines the initial position of the hose. Subsequent measurements reveal the differences in height.

3.3 Pore pressures

Pore pressures have been measured between April and November 2000. Data are visualized in Figure 3.4. The transducers that still worked reliably in August 2000, are located in section line B, near the location of the Begemann boring. They are stacked vertically, their initial depth is listed in Table 3.3. Transducers were usually read twice a day, at 6h and at 18h. When a load step was to be added, pore pressures were read every hour. Each day data was transmitted automatically to GeoDelft.

Pore pressure transducer	Depth [m]	End date
14689	-3.562	2000-05-03
14736	-5.033	2000-11-23
18010	-8.107	2000-08-02
18019	-12.147	2000-07-13

Table 3.3 Pore pressure transducers and their depth.

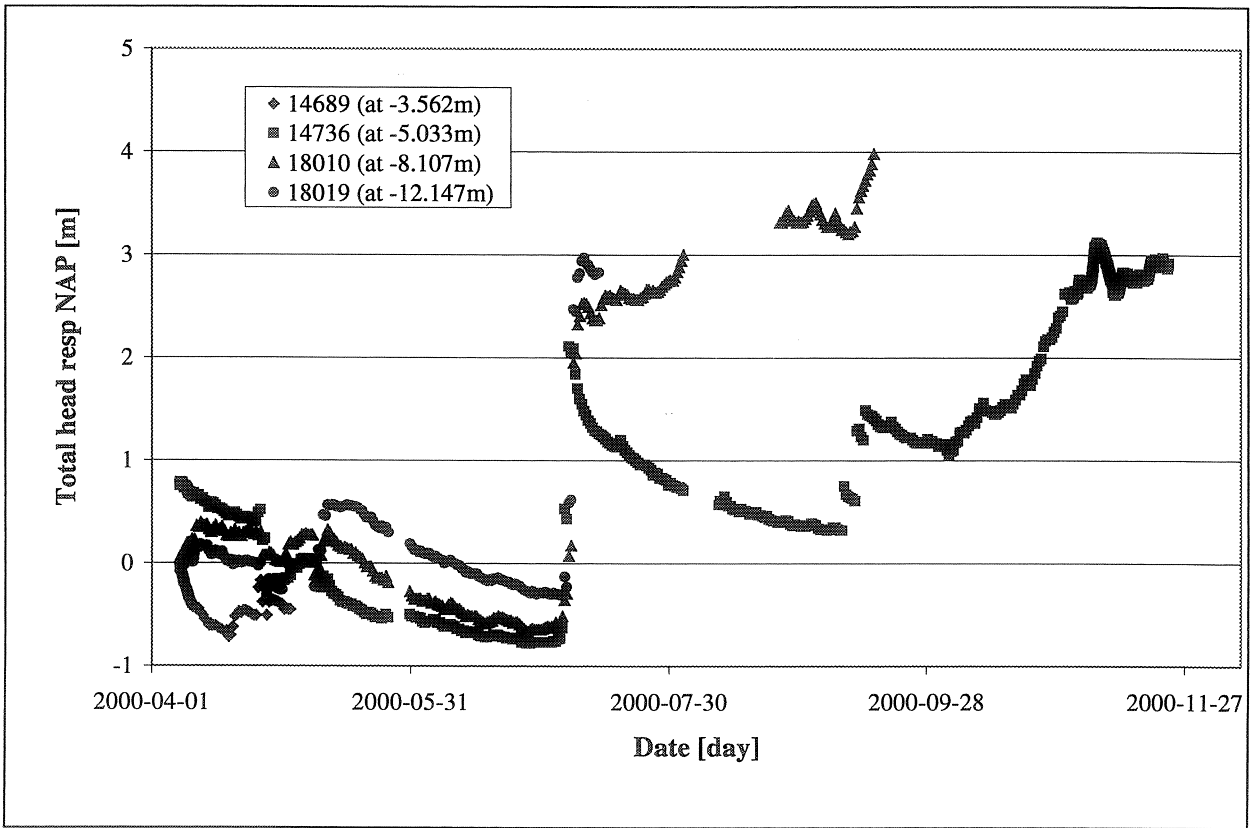

Figure 3.4 Total head measured by the pore pressure transducers

3.4 Lab tests

In both section lines "raai A" and "raai B" a Begemann boring was made. The core in section A was slightly damaged, which made it less suitable for K_0 tests. Samples from the core in section line B were tested.

3.4.1 K_0-CRS test

The K_0-CRS test is described in more detail in report [den Adel 2002]. Six samples were tested, see Table 3.4,

Material	a	b	c
Dunkirque clay	0.0049	0.0566	0.0028
Holland peat	0.035	0.276	0.0217
Gorkum heavy 1	0.0123	0.1374	0.0061
Gorkum light	0.0203	0.1765	0.0138
Gorkum heavy 2	0.0103	0.1130	0.0057
Base peat	0.0134 [1]	0.1344	0.0127

Table 3.4 Samples tested in the K_0 equipment

3.4.2 Compression test

Six samples were tested, see Table 3.5 and Table 3.6.
The preconsolidation stress p_g is the intersection of the tangents of the first and last part of the curve $\varepsilon^H - \ln \sigma_v'$. The permeability is determined using the Taylor method.

Material	σ'_v [kPa]	p_g [kPa]	OCR
Dunkirque clay	9.8	47	3 [2]
Holland peat	31.24	51	1.63
Gorkum heavy 1	38	52	1.37
Gorkum light	45.8	85	1.86
Gorkum heavy 2	52.9	110	2.08
Base peat	-	-	-

Table 3.5 The effective stress, limit stress and over consolidation ratio as measured in the compression test

Material	vertical permeability [m/s]	horizontal permeability [m/s]	γ_{wet} [kN/m^3]	γ_{dry} [kN/m^3]	porosity [-]
Dunkirque clay	5.53×10^{-10}	5.53×10^{-10}	19.2	14.5	0.5
Holland peat	2.31×10^{-8}	9.24×10^{-8}	10.4	1.7	0.9
Gorkum heavy 1	8.95×10^{-9}	8.95×10^{-9}	16.0	9.7	0.6
Gorkum light	1.59×10^{-8}	1.59×10^{-8}	13.0	5.8	0.7
Gorkum heavy 2	8.95×10^{-9}	8.95×10^{-9}	16.7	11.2	0.6
Base peat	8.99×10^{-9}	3.6×10^{-8}	12.0	4.0	0.8

Table 3.6 Permeability as measured in the compression test

The anisotropy in the permeability of Holland peat is an assumption, based on experience.

1 10% of the value of b
2 The actual value is 4.8. Experience has brought us the knowledge that this is a value that is too high. Thus a value of 3 is chosen.

3.5 Re-analysis of data

3.5.1 a and b

In the spring of 2002 the program MCompress has been developed. Its aim is to evaluate the results of oedometer tests and K_0 tests. The data of the K_0 tests, as summarized in section 3.4.1, have been reanalyzed using MCompress. The results are shown in Table 3.7.

Material	a MCompress	b MCompress	a Manual	b Manual
Dunkirque clay	-	-	0.0049	0.0566
Holland peat	0.0348	0.275	0.035	0.276
Holland peat	0.0350	0.356	0.0344	0.352
Gorkum heavy 1	0.0123	0.134	0.0123	0.1374
Gorkum light	0.0206	0.180	0.0203	0.1765
Gorkum heavy 2	0.0102	0.109	0.0103	0.1130
Base peat	-	-	0.0134	0.1344

Table 3.7 Values of a and b as determined with MCompress and manually

When we compare the data from [den Adel 2002] with the data from Table 3.7, there are no large differences. Since the c value was determined from oedometer test data, these values are unchanged with respect to report [den Adel 2002].

3.5.2 Pre-overburden pressure

The pre-overburden pressure (POP) is defined as:

$$POP = p_g - \sigma'_v \tag{3.1}$$

where p_g is the preconsolidation stress and σ_v' is the actual effective vertical stress. Both are determined in a sample.

The data of the K_0 tests have been reanalyzed in a spreadsheet to determine the p_g value. For the interpretation, the Casagrande method is used, as shown in Figure 3.5: point A is the point where the curvature is maximal, AB is the tangent and EF is the final slope of the curve.
The usual Casagrande method uses linear strain but, as the Isotache model is intrinsically linked to natural strain, a second interpretation with natural strain was performed. The results of both interpretations with different definitions for the strain, are shown in Table 3.8. The difference between the values of p_g is less than 11%.

Material	p_g [kPa]		Relative error [%]
	Linear strain	Natural strain	
Dunkirque clay	53.8	60.5	11.1
Holland peat	61.0	66.7	8.5
Gorkum heavy 1	59.3	62.3	4.8
Gorkum light	75.1	84.7	11.3
Gorkum heavy 2	125.1	132.9	5.9

Table 3.8 The pre-consolidation pressure as determined manually from the K_0-CRS using linear and natural strain

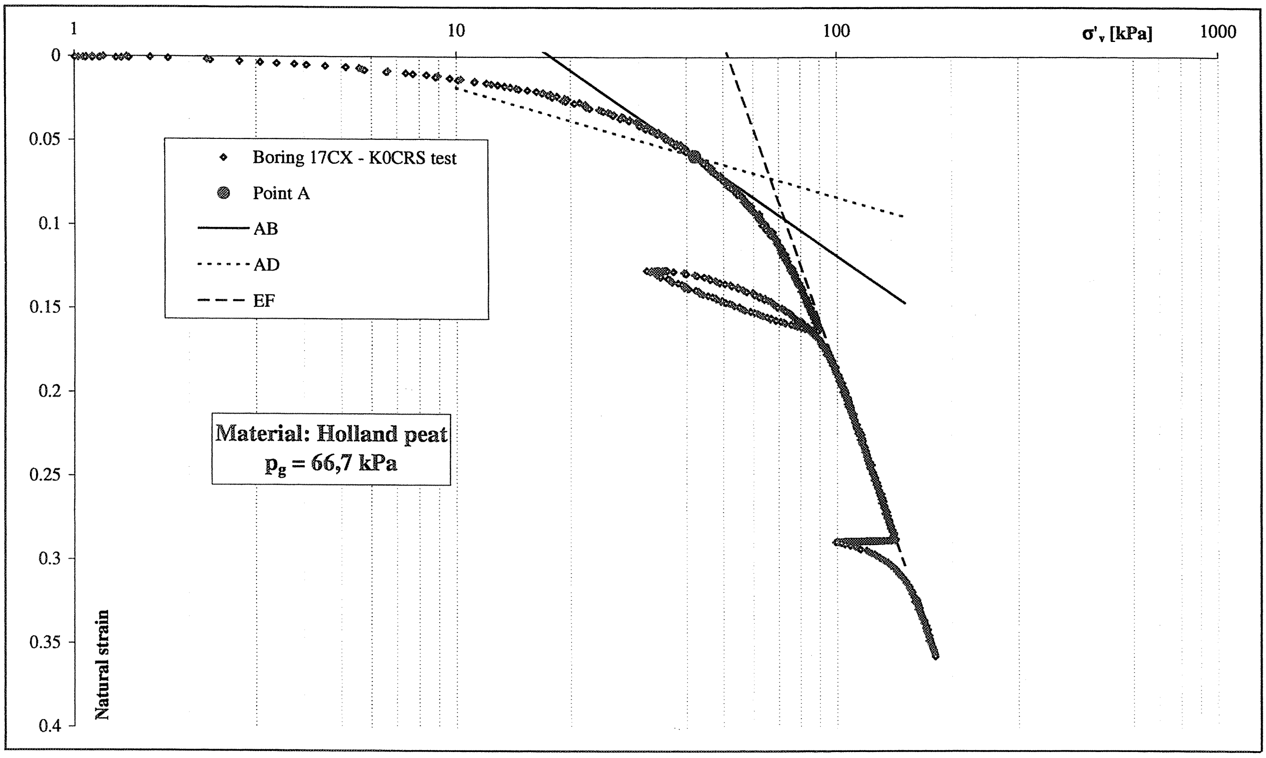

Figure 3.5 *Determination of the preconsolidation stress p_g from the K_0-CRS test*

The values of the p_g as determined from the K_0-CRS tests using natural strain are compared with those from the previous compression tests (see Table 3.5) in Table 3.9. The difference can reach 30%, as the interpretation from a compression test uses only a few points, whereas the interpretation from a K_0-CRS test uses several hundreds of points. For this reason, we will use the p_g values from the K_0-CRS test for the calculations in section 4.

Material	p_g [kPa]		Relative error [%]
	Compression test	K_0-CRS test	
Dunkirque clay	47	60.5	28.7
Holland peat	51	66.7	30.8
Gorkum heavy 1	52	62.3	19.8
Gorkum light	85	84.7	0.4
Gorkum heavy 2	110	132.9	20.8

Table 3.9 *The pre-consolidation pressure as determined from the K_0-CRS and the compression tests*

The POP values as used in the calculations in section 4, are shown in Table 3.10. The values of the POP are assumed to be constant for a specific material and thus for the corresponding layer.

Material	σ'$_v$ [kPa]	p_g [kPa]	POP [kPa]
Dunkirque clay	14.8	60.5	45.7
Holland peat	32	66.7	34.7
Gorkum heavy 1	36	62.3	26.3
Gorkum light	43.7	84.7	41
Gorkum heavy 2	50.4	132.9	82.5
Base peat	-	-	58.6 [3]

Table 3.10 The effective stress, pre-consolidation stress and pre-overburden pressure

3.5.3 Permeability

In 2002 a strain dependent permeability was introduced in MSettle. This feature is not yet present in MCompress. The analysis of the measured data was done by means of a spreadsheet, see Figure 3.6.

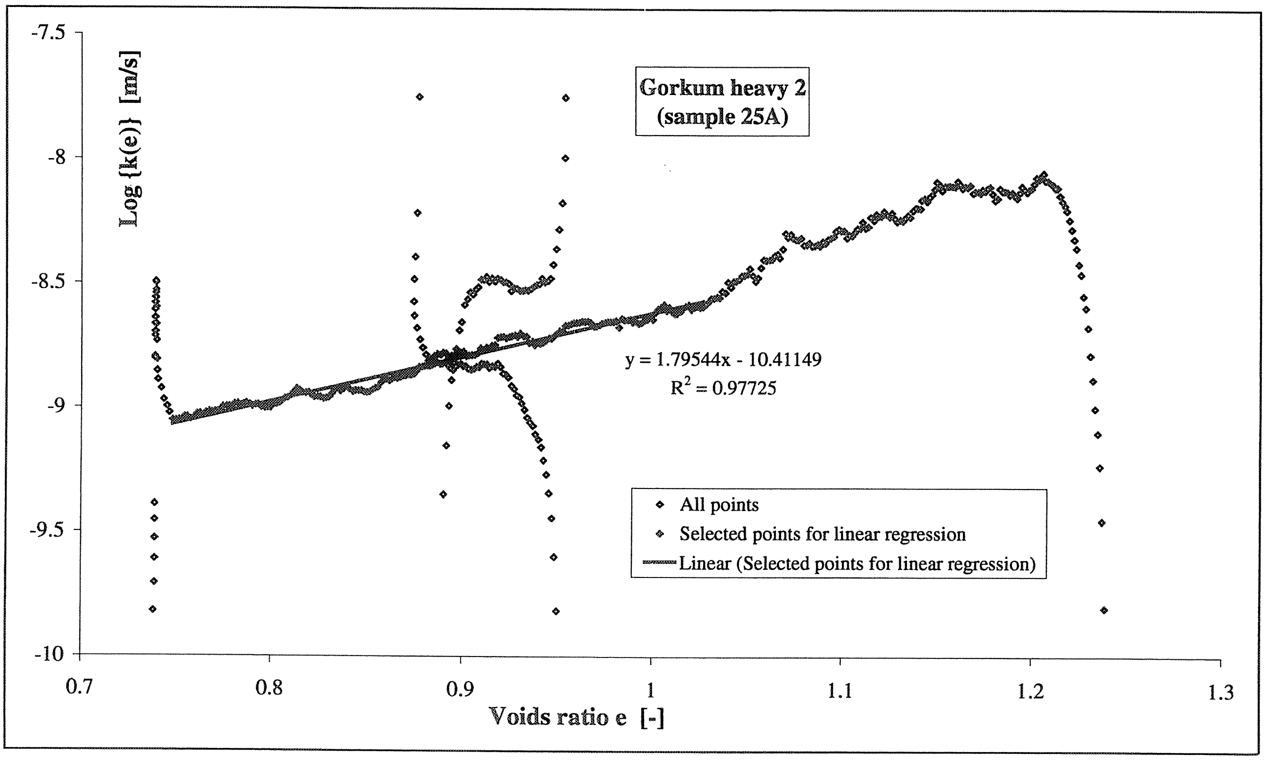

Figure 3.6 Determination of the strain dependent permeability

The results of the analysis for all samples, comparable to Figure 3.6, are shown in Table 3.11.

[3] This value is the average of Gorkum heavy 2 and Holland peat.

Material	Vertical permeability $k(e_0)$			Void ratio e_0	C_{ke}
		[m/s]	[m/d]	[-]	[-]
Dunkirque clay	Strain dependent	1.255×10^{-8}	1.084×10^{-3}	0.89	0.207
Holland peat		2.922×10^{-8}	2.525×10^{-3}	5.76	2.605
Gorkum heavy 1		1.243×10^{-9}	1.074×10^{-4}	1.46	0.987
Gorkum light		3.643×10^{-9}	3.148×10^{-4}	2.72	1.065
Gorkum heavy 2		6.529×10^{-9}	5.641×10^{-4}	1.24	0.557
Base peat	Constant	8.981×10^{-9}	7.760×10^{-4}	4.56	∞

Table 3.11 Determination of strain dependent permeability parameters (manually)

For the meaning of $k(e_0)$, e_0 and C_{ke} is referred to equation (2.5). The value of e_0 is the initial void ratio, as measured in the K_0-CRS experiment.

The measured value of the permeability at void ratio 0.8, k(0.8), equals 10^{-9} m/s. The value calculated with equation (2.5) equals 1.06×10^{-9} m/s. In Figure 3.6 several strings of points can be distinguished. The vertically oriented strings originate from unloading and reloading phases in the experiment. The more horizontally oriented strings are obtained during continuous sections in the loading scheme.

In Table 3.12 the values of the vertical permeability as determined from the K_0-CRS tests using natural strain are compared with those from compression tests using Taylor method (see [den Adel 2002]). After re-calculation, the permeability as determined from the compression test for the Dunkirque clay is about 1.35×10^{-8} m/s instead of 5.53×10^{-10} m/s, as reported in [den Adel 2002]. The relative difference between the permeabilities for the Dunkurque clay, as determined from the K_0-CRS test and the recalculation decreases to only 7%. However, the observed relative differences can reach nearly 90%.

Material	$k(e_0)$ [m/s]		Relative error [%]
	Compression test (Taylor method)	K_0-CRS test (strain dependent)	
Dunkirque clay	5.53×10^{-10}	1.255×10^{-8}	2169
Holland peat	2.31×10^{-8}	2.922×10^{-8}	26
Gorkum heavy 1	8.95×10^{-9}	1.243×10^{-9}	86
Gorkum light	1.59×10^{-8}	3.643×10^{-9}	77
Gorkum heavy 2	8.95×10^{-9}	6.529×10^{-9}	27
Base peat	8.99×10^{-9}	-	-

Table 3.12 The vertical permeability as determined from the K_0-CRS and the compression tests

3.5.4 Piezometric line

Using the data in Figure 3.4 it can be seen that the piezometric head, as used in report [den Adel 2002] for all layers is too low: -1.3 m. The graph points a value of roughly -0.7 m.

3.5.5 Vertical drains

In the input for the calculations the bottom position of the vertical drains was changed. In [den Adel 2002] it was on -12.9m, extending into the Pleistocene sand. Such a configuration is not sound, since water will squirt out of the drain. Usually an end position of approximately 2 m above the Pleistocene sand is chosen. So we reduced the end position of the vertical drains to -10m NAP. Furthermore the total head in the Pleistocene sand is changed from -1.3m to -0.7 m.

3.6 Sill stress

The artificial sill stress was introduced [Sellmeijer 2002] in order to quench numerical oscillations. Analysis of the data of the "Waardse Alliantie" [den Haan 2002] has shown that the sill stress has an unacceptable large influence on the settlement. In the previous calculations, the sill stress was set to 25 kPa. The sill stress has been replaced by the limit stress. In the calculations for this report, the limit stress is set to 0.

4 Calculations

Experimental data, as presented in section 3, has been used in the calculations. The Isotach model has been programmed in the MSettle suite. Version 6.7 has been used. Several cases have been calculated, listed in Table 4.1:

Nr	Upper geometry	γ_{dry} [kN/m^3]	Remarks
1	GeoDelft	18	Basic situation for GeoDelfts geometry
2	Contractor	18	Basic situation for contractors geometry
3	GeoDelft	17	Sand is slightly drier
4	Contractor	17	Sand is slightly drier
5	GeoDelft	18	Without loading step at day 332
6	GeoDelft	18	p_g is determined using linear strain

Table 4.1 Cases

The reason for the six cases is to visualize the effect that uncertainties in soil properties and in boundary conditions have on settlement.

Differences in the results of cases 1 and 2 illustrate the sensitivity of the calculated settlement for minor deviations in the upper geometry.
Differences between cases 1 and 3 and between cases 2 and 4 illustrate the effect of partial saturation of the soil above the water table, e.g. by periods of drought or of heavy downpour.
Case 5 illustrates yet another effect of variations in the upper geometry. In the data as compiled by the contractor, at day 332 a minor peak can be seen in the external boundary condition. An extra load of 0.35m of sand was added. As discussed in section 3.1.2 there are some doubts.
Case 6 illustrates the effect of the type of strain (linear or natural) used in the Casagrande interpretation for the determination of the preconsolidation stress p_g.

Cases 1 and 2 correspond to the most likely parameters and geometry, and can be seen as the best guess.

The results of the calculations are shown in Figure 4.1. The experimental data are drawn as separate markers. Calculations are displayed as lines. The number of the cases refers to the cases listed in Table 4.1.

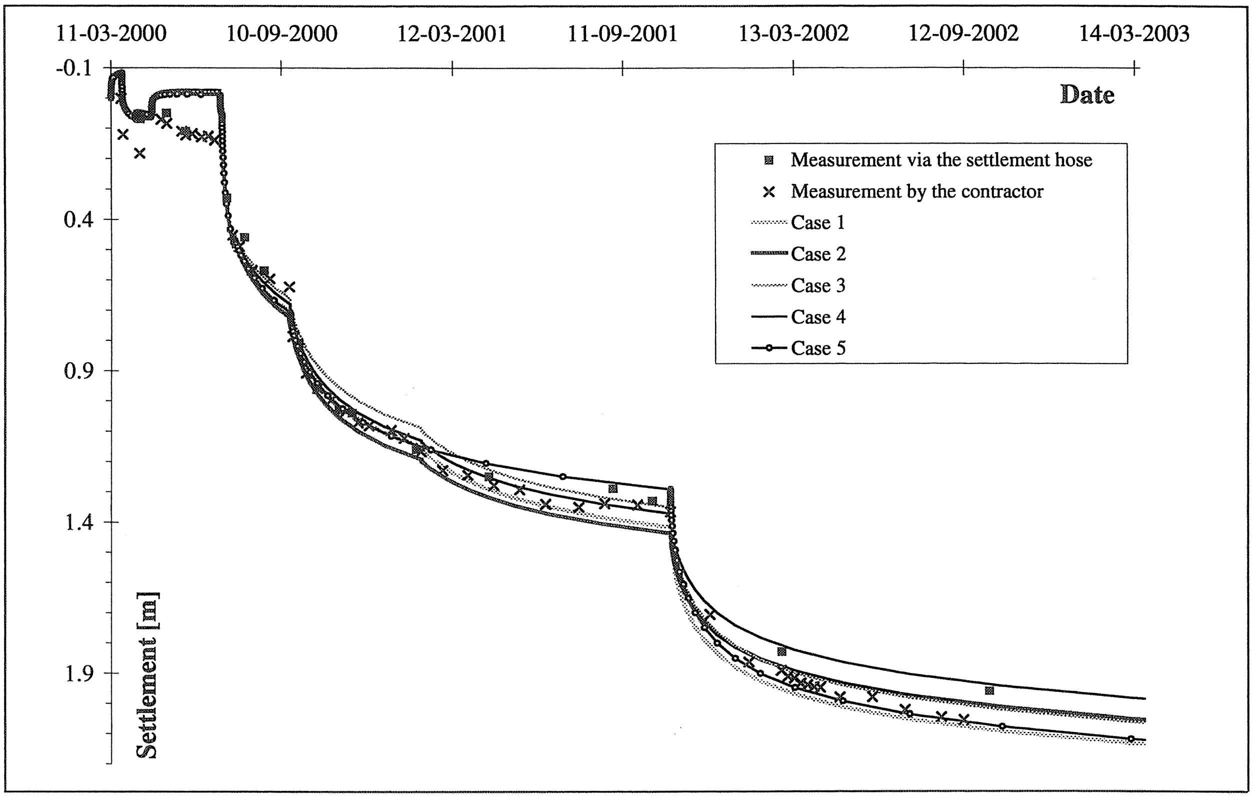

Figure 4.1 *Postdictions and measured settlements for cases with an uncertainty in mechanical properties*

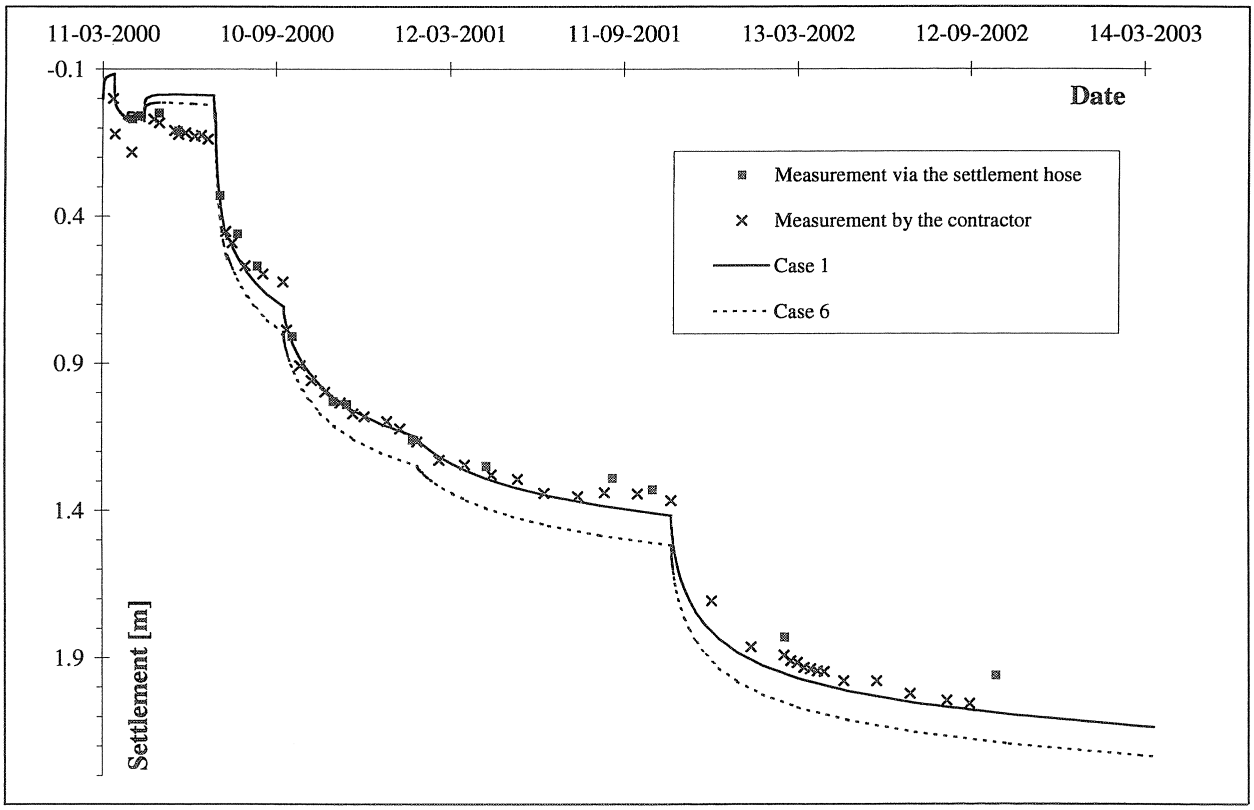

Figure 4.2 *Postdictions and measured settlements for cases with a different preconsolidation stress*

Next to these cases, some additional calculations were performed, in which the date the vertical drainage has been installed, has been varied. Although there is not much doubt about the date, an absolute guarantee can not be given. Therefore results have been calculated when vertical drainage was installed after 12 and after 39 days. There is an effect visible at the beginning of the activities, so a section of Figure 4.1 is shown in Figure 4.3.

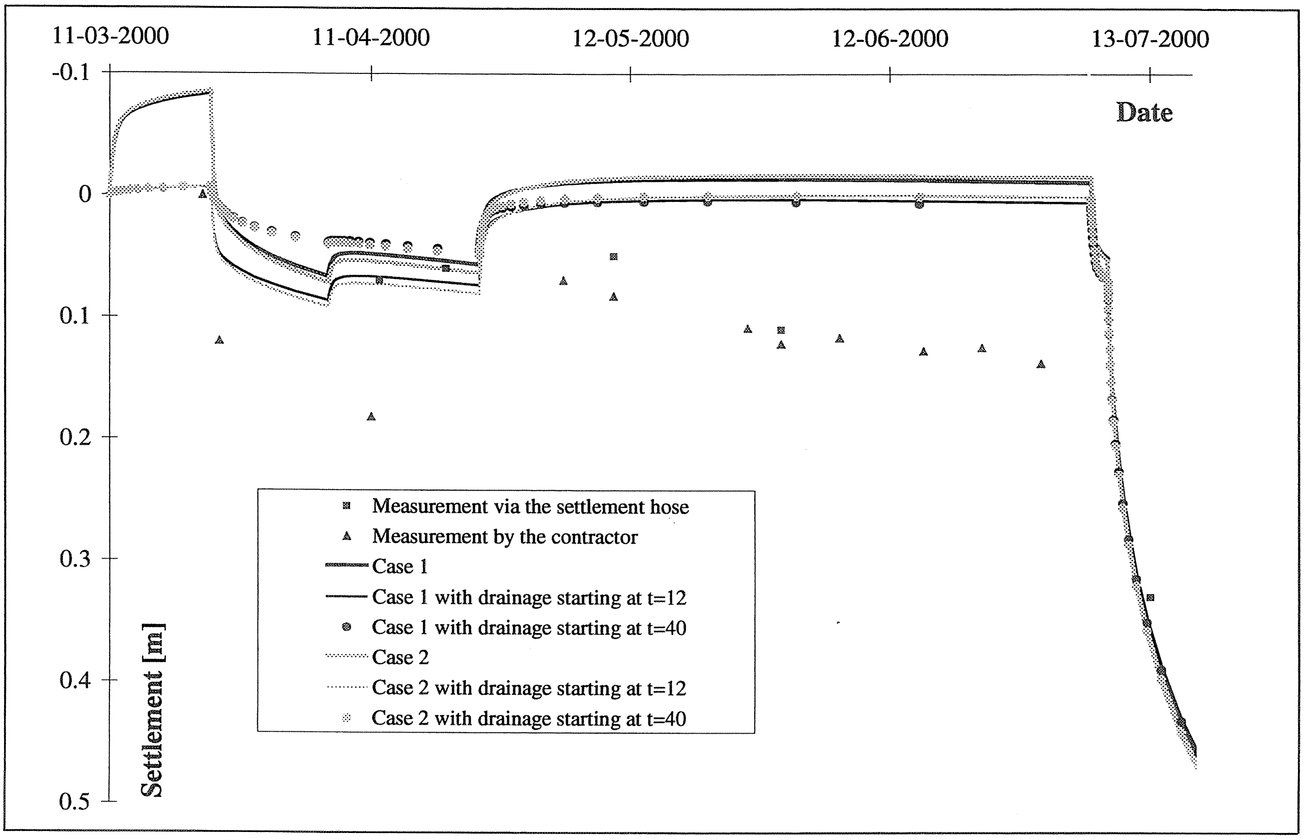

Figure 4.3 *The effect of the date the vertical drainage was installed.*

Finally the calculated pore pressures are compared with the measured pore pressures, see Figure 4.4. While the settlement is susceptible for both the mechanical and the hydraulic properties of the subsoil, the pore pressures are mainly susceptible for the hydraulic properties of the subsoil. So the pore pressures provide the opportunity to check the validity of just one parameter of the set of parameters that control the settlement process, independent of the mechanical properties.

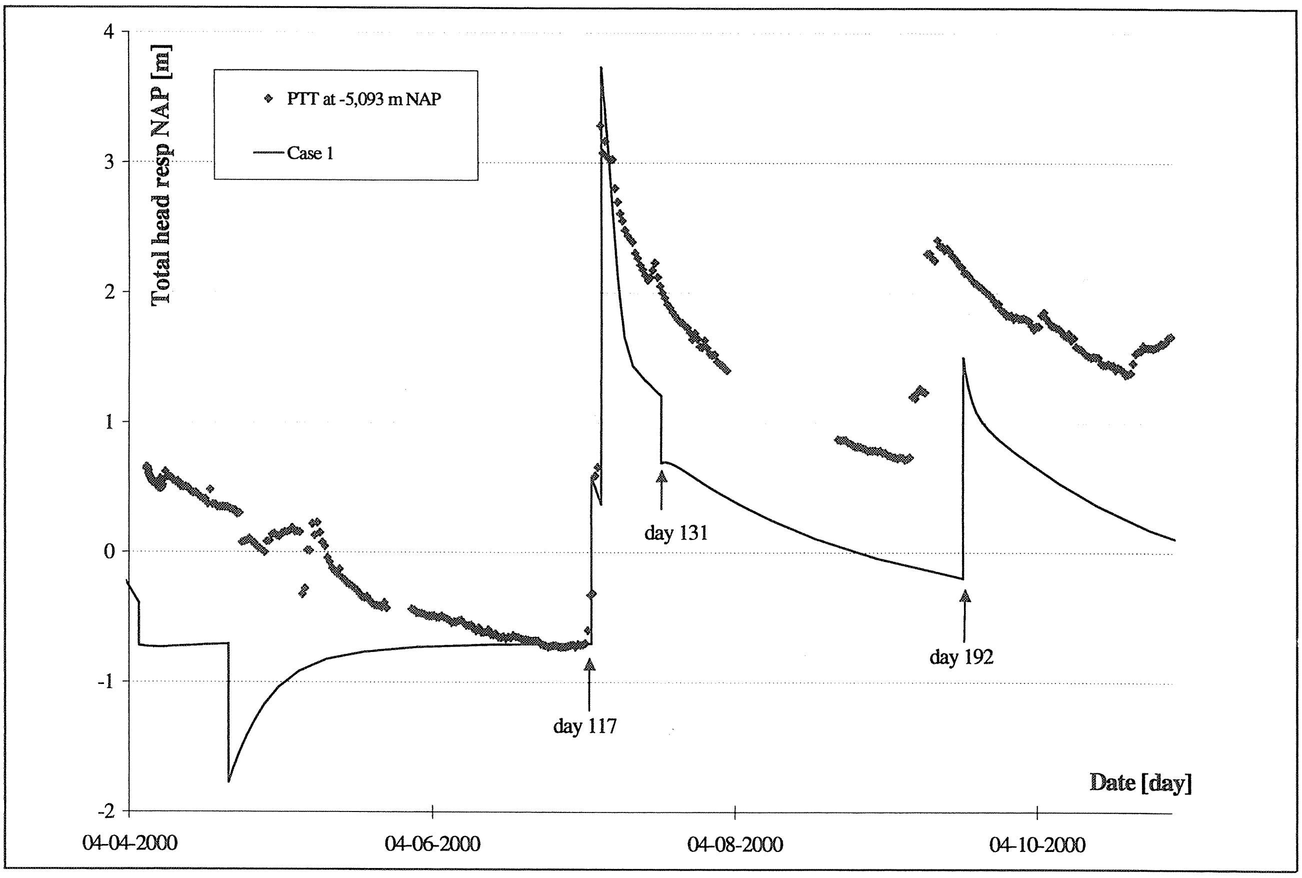

Figure 4.4 *The calculated and measured pore pressures in section line "Raai B"*

The excavation and heightening on day 131 are virtual actions, as needed in the calculations to simulate the fact that within two weeks after applying the sand water mixture, the water has drained. Drainage is assumed to reduce the volumetric weight of the sand from 20 to 18 kN/m^3.

The increment in pore pressure as calculated on day 192 does not coincide with the observed increase in pore pressure. The automatically sampled pore pressures suggest that the contractor has started to supply sand two weeks earlier than reported in the logs.

5 Discussion

5.1 Settlement

In Figure 4.1 it can be clearly seen that the correlation between postdiction and measured settlements is very good, particularly for case 1 which uses the more probable geometry and parameters. The family of settlement curves constituted by the different cases includes the measurements except for the first load-step. Note that this agreement is obtained using the data as measured in the laboratory, without tinkering with the measured parameters. This means that the Isotach model is a sound schematization of the processes consolidation and creep, as observed in the field. Furthermore it means that the K_0-CRS test is a sound test to determine the parameters, which predominantly control settlement, by elastic and plastic strain as well as by creep.

Another striking observation is that the calculated settlement is susceptible for variations in the geometry of the load. As can be seen in Figure 3.1 and Figure 3.2, there are small differences in geometry. The location where the contractor has measured his geometry is very close to the location GeoDelft has measured the geometry; however, local differences may occur. Although the differences are small, they lead to variations in the calculated settlement of the order of roughly 0.1 m.

Differences in geometry originate from the method it is determined:

- The contractor determines the outer geometry in a section line on the same spot where the three settlement plates are located. The outer plates give hardly any information, the height is more or less constant, only the central plate provides increasing heights. The resulting embankment is more or less a triangle or a trapezium, if an assumption is made for the slope. The contractor has measured the height of the embankment more frequently than GeoDelft.
- GeoDelft has determined the outer geometry in more detail. Different heights in a section line are clearly visible. GeoDelft has measured the outer shape of the embankment at random moments, usually before or after an increment in loading.

Since small variations in load produce relative differences in the settlement of the order of 5%, the geometry of the load should be known in detail if the settlement is to be known in great detail as well.

Variations in the volumetric weight of the load produce relative variations of the order of 5% as well. The volumetric weight of the load is usually assumed to be 18 kN/m^3. Measurements of the volumetric weight are not present. There are two reasons why the volumetric weight might vary.

- Variations in water content. The more water is present, the higher the volumetric weight will be, leading to a larger settlement. Water content will vary due to drainage of the sand water mixture and due to seasonal influences in precipitation. The water content of naturally drained sand depends on the grain size distribution. Presence of a high fraction of fines produces a higher water content than when fines are absent.
- The density of the load. If the load is compacted, the volumetric weight is less susceptible to variations in water content.

Although the calculated overall settlement after roughly one month is in good agreement with the measured settlement, in the initial stages, when part of the terrain has been excavated and lateral embankments have been erected, the agreement between calculation and measurement is not so good, see Figure 4.3. Unloading and reloading provides deviations between calculation and measurement. Nevertheless after two months, when substantial loading has taken place, the agreement improves significantly.

In the initial stages the observed variations in settlement are smaller than calculated. Since the deviations between measurement and calculation occur at low stresses, the argument may be raised that the POP concept ruins the agreement. However, this argument can be denied. By adding the POP to the effective stress, variations in effective stress are relatively lower than without POP. As a result one may conclude that the calculated fluctuations should be less than observed. The opposite is the case, thus contradicting the assumption.

The influence of the date vertical drainage started, on settlement is small, as can be seen in Figure 4.3. The main influence can be seen in the early stages of the project, when the central section was excavated. Since no abundance of measured data for the settlement is present, no distinction can be made to discriminate between day 0 and day 12. The value as determined by the contractor on 2000-03-22 points to a date of placement of at least day 12. The calculation, which assumes day 39, is in good agreement with the measured data.

In Figure 4.2 it can be clearly seen that the agreement between measurement and calculation is slightly better when the preconsolidation stress is determined with an adapted Casagrande method, that uses natural strain instead of the usual linear strain. Variations in the calculated settlement are of the order of roughly 0.1 m. As the Isotach model uses natural strain, this adapted method seams more obvious.

5.2 Pore pressures

Pore pressures of the transducer 14796 at a depth of -5 m NAP are shown in Figure 4.4. After the increment in height has been made (day 117), the pore pressure rises sharply. Thereafter the excess pore pressures start to dissipate. The sharp decrease in pore pressure at day 131 originates from the artificial load step to simulate drainage of the water in the load. It is artificial, since the water drains from the load in a more continuous way than instantaneously as used in the calculations. As a matter of fact, it starts draining just at the day the load was added. A continuous draining of the load is not (yet) possible in MSettle. The overall shape of the calculated dissipation of the pore pressure is reasonably in agreement with the experimental data. The same holds for the increment in load at day 192. Although this load will have been incremented some ten days earlier than specified, the shape of both calculated and measured pore pressures are similar. The predicted step in pore pressure is the same as has been measured. So the load increment as used in the calculations, is in agreement with the actual load step. The absolute value of the predicted height of the pore pressure is roughly 1 m lower than measured. A more careful examination of the slope further reveals that the calculated slope is slightly steeper than the slope for the measured pore pressures.

5.3 Improvement

The observed excess pore pressures seem to dissipate somewhat slower than calculated. Combined with the facts mentioned in sections 5.2 this leads to a conclusion that the permeability in the calculations should be somewhat lower than measured in the laboratory. When the permeability in the calculations is assumed to be lower than in case 1, the rate the excess pore pressure dissipates, is lower. The pore pressure transducer 14796 is located in the layer of Gorkum heavy 1 and, according to Table 3.12, the method by which the permeability of this material is determined, can lead to deviations of about 90%. This is an indication for the difficulty in the determination of this parameter. In order to visualize its influence an additional calculation has been performed. Basically it is the same as case 1. The only difference is that the permeability of all layers of soil is 30% of the permeability as listed in Table 3.11. The results are presented in Figure 5.1. The pore pressure predicted by MSettle now agrees better with the experimental data than when the 100% value of the permeability is used.

Changing the permeability has consequences for the settlement as well. These consequences are shown in Figure 5.2. The correlation between calculated and measured settlement is then not as good as previously but good enough since uncertainties in the shape and the height of the external load exist. Moreover, the adaptation of 30% of the value of the permeability is done for all the layers. This is probably not the best assumption but, since only one transducer has reliable pore pressure measurements, we can't check it for the other layers.

The permeability is probably the most difficult parameter to evaluate correctly. Such difficulties were also found in the re-analysis of "Waardse Alliantie" project.

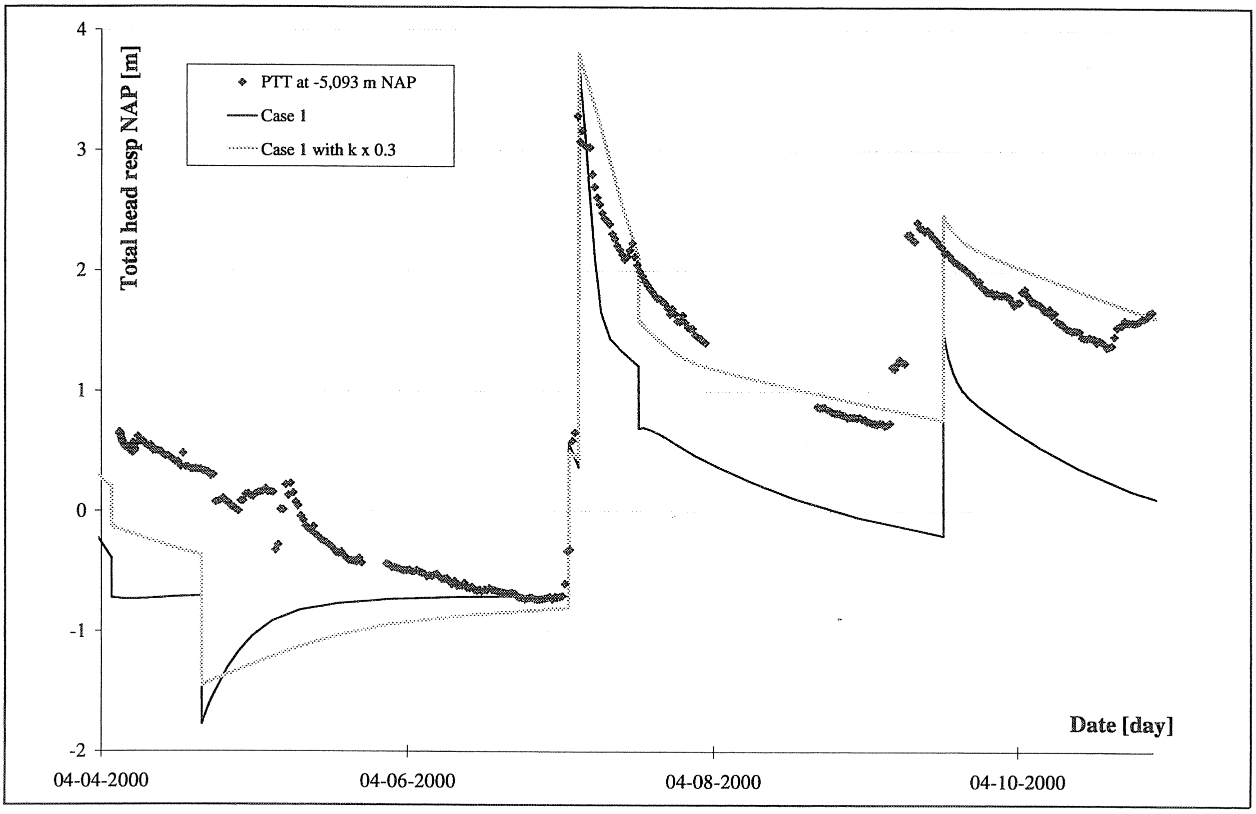

Figure 5.1 *Influence of the permeability on the pore pressures*

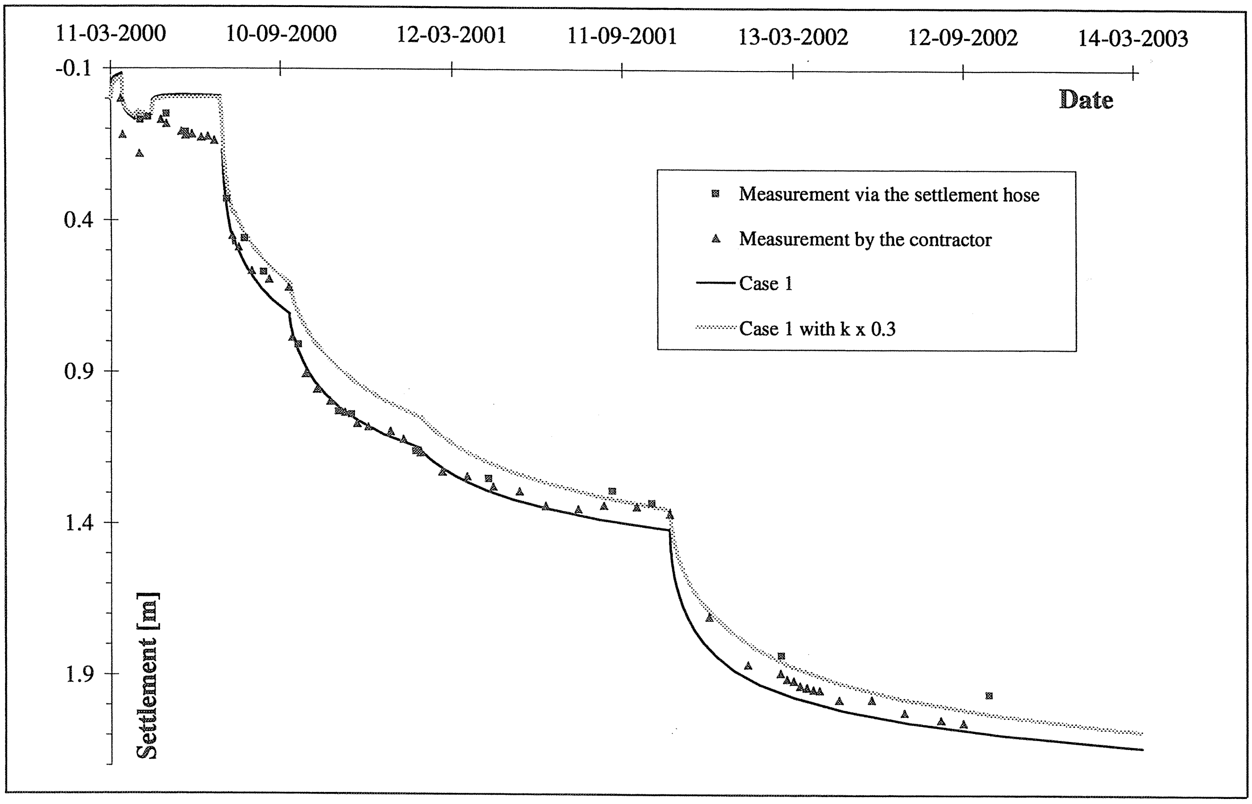

Figure 5.2 The consequences of the 30% permeability on settlement

6 Conclusions

The calculations performed for the "Barendrechtse weg" show that the agreement between calculated and measured settlements is very good. This result is obtained using the mechanical and hydraulical parameters of the sub soil, as determined in the laboratory. As a result it should be concluded that the combination of the Isotach model and the K_0-CRS tests is a sound method to predict settlement due to the construction of an embankment.

The concept of the strain dependent permeability, as introduced earlier in this project ("Waardse Alliantie"), is also found to be a sound concept.

Unloading and reloading in the initial phase of the project, when changes in the effective stress are high compared to the prevailing effective stress, is not so adequately well predicted by the model. The calculated variations are higher than observed. It is shown that the POP concept in the Isotach model can not be responsible for these deviations.

Fluctuations in the shape and height of the external load do influence the calculated settlement. So when accurate predictions of the settlement are needed, the external load should be known in detail. When the shape and height of a road under construction differs from the shape and height, as prescribed in the detailed design, this will lead to significant deviations from the calculated settlement. A similar argument holds for the volumetric weight of the material to be used in a road under construction.

The pore pressures measured in one layer are higher than predicted. Furthermore they tend to dissipate slower than calculated. This leads to the conclusion that the permeability of this layer, as determined in the laboratory, is too high. If the permeability is reduced to 30% of the permeability, as measured in the laboratory, the agreement between experimental pore pressure data and the calculated values is better.

However, the agreement between calculated and measured settlement becomes less good if the permeability of the soil is assumed to be roughly 3 times lower than found in the K_0-CRS experiments. The factor 3 is probably not applicable for all the layers but, as only one transducer has relevant pore pressure measurements, the evaluation of the correct factor for each layer is not possible.

An integrated approach of settlement and pore pressures provides more detailed information and leads to a prediction of settlement with an enhanced accuracy on condition that quality of measurements is pertinent.

7 Recommendations

In this analysis three items have been found, which influence the agreement between measured and calculated properties. Since these items will influence the prediction of settlement, special attention should be paid to these items, if an accurate prediction of settlement is needed.

The height and shape of an embankment under construction should be known in detail, if accurate predictions of settlement are needed.

The volumetric weight of the material in the embankment should be known in detail, if an accurate prediction of settlement is needed.

Monitoring the pore pressures in the sub soil, in the centre of the embankment provides additional information on the permeability of the sub soil. It allows for an appropriate tuning of the permeability as measured in the laboratory. This may enhance the reliability of the prediction of the settlement.

When increasing the height of an embankment using dredging techniques, the embankment itself has to drain. This process can not yet be modelled in he MSettle suite in an elegant way. A simple model for draining the embankment should be added to MSettle.

Stress dependent permeabilities can not yet be calculated by MCompress. Calculation by means of a spreadsheet does provide the information needed; however, the method is susceptible to errors, which are erroneously introduced by inexperienced users. In order to prevent these errors, the method for the determination of the stress dependent permeability should be added to MCompress.

Determination of the preconsolidation stress from a compression test using the Casagrande method has a higher uncertainty than the values as obtained from a K_0-CRS test, since the number of regression points is rather small. Moreover, an adapted Casagrande method that uses natural strain instead of the usual linear strain is more obvious as the Isotach model uses natural strain as well. Preconsolidation pressure can not yet be calculated by MCompress but it should be added.

Since uncertainty in the geotechnical parameters of the sub soil will always be present, its influence on the predicted settlement has to be accounted for. Since the influence of this uncertainty is unknown, it should be investigated. Such an analysis will lead to the information which sub soil parameter of the Isotach model has a dominant effect on the predicted settlement. If the influence of this parameter leads to an unacceptable uncertainty in the calculated settlement, the natural variation of the according parameter in the sub soil has to be determined, in order to determine the relevance of a more accurate characterisation of the sub soil.

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General Appendix: Delft Cluster Research Programme Information

This publication is a result of the Delft Cluster research-program 1999-2002 (ICES-KIS-II), that consists of 7 research themes:
▶ Soil and structures, ▶ Risks due to flooding, ▶ Coast and river, ▶ Urban infrastructure,
▶ Subsurface management, ▶ Integrated water resources management, ▶ Knowledge management.

This publication is part of:

Research Theme	:	Soil and structures		
Baseproject name	:	Wegen en spoorwegen		
Project name	:	Samengestelde constructies		
Projectleader/Institute		Dr. H. den Adel		GeoDelft
Project number	:	01.04.02		
Projectduration	:	01-03-2000	-	31-03-2003
Financial sponsor(s)	:	Delft Cluster		
		GeoDelft		
		TNO		
		RWS Dienst Weg- en Waterbouwkunde		
		TUD		
		Stybenex		
		Waardse Alliantie		
Projectparticipants	:	GeoDelft		
		TUD		
		TNO		
Total Project-budget	:	€ 873.000		
Number of involved PhD-students	:	1		
Number of involved PostDocs	:	0		

Delft Cluster is an open knowledge network of five Delft-based institutes for long-term fundamental strategic research focussed on the sustainable development of densely populated delta areas.

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Other Involved personnel

The realisation of this report involved:

Name	**Organisation**
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2 A. L. Schapers	GeoDelft

ANNEXES
Annex 1 Input MSettle, case 1

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         10     11
     13 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         11     12
     14 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         17      8
     15 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         11     18
     16 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         15      7
     17 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         12     16
     18 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
          3      4
     19 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
          5      6
     20 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         35     36
     21 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
          1      2
     22 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
          2      7
     23 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         12     13
     24 - Curve number
        2 - number of points on curve,  next line(s) are pointnumbers
         13     14

[BOUNDARIES]
   12 - Number of boundaries -
      0 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
         20
      1 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
          1
      2 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
          2
      3 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
          5
      4 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
          4
      5 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
          3
      6 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
          6
      7 - Boundary number
        1 - number of curves on boundary, next line(s) are curvenumbers
```

```
        7
    8 - Boundary number
      1 - number of curves on boundary, next line(s) are curvenumbers
        8
    9 - Boundary number
      5 - number of curves on boundary, next line(s) are curvenumbers
       14     10     11     12     15
   10 - Boundary number
      7 - number of curves on boundary, next line(s) are curvenumbers
       16      9     10     11     12     13     17
   11 - Boundary number
      9 - number of curves on boundary, next line(s) are curvenumbers
       21     22      9     10     11     12     13     23     24

[PIEZO LINES]
  2 - Number of piezometric level lines -
    1 - PlLine number
      1 - number of curves on PlLine,  next line(s) are curvenumbers
       18
    2 - PlLine number
      1 - number of curves on PlLine,  next line(s) are curvenumbers
       19

[PHREATIC LINE]
  1 - Number of the piezometric level line acting as phreatic line -

[WORLD CO-ORDINATES]
          0.000 - X world 1 -
          0.000 - Y world 1 -
          0.000 - X world 2 -
          0.000 - Y world 2 -

[LAYERS]
 11 - Number of layers -
    1 - Layer number, next line is material of layer
      Basisveen
      2 - Piezometric level line at top of layer
      2 - Piezometric level line at bottom of layer
      1 - Boundarynumber at top of layer
      0 - Boundarynumber at bottom of layer
    2 - Layer number, next line is material of layer
      Gorkum zwaar 1
     99 - Piezometric level line at top of layer
     99 - Piezometric level line at bottom of layer
      2 - Boundarynumber at top of layer
      1 - Boundarynumber at bottom of layer
    3 - Layer number, next line is material of layer
      Gorkum zwaar 2
     99 - Piezometric level line at top of layer
     99 - Piezometric level line at bottom of layer
      3 - Boundarynumber at top of layer
      2 - Boundarynumber at bottom of layer
    4 - Layer number, next line is material of layer
      Gorkum licht
     99 - Piezometric level line at top of layer
     99 - Piezometric level line at bottom of layer
      4 - Boundarynumber at top of layer
      3 - Boundarynumber at bottom of layer
    5 - Layer number, next line is material of layer
      Gorkum zwaar 2
     99 - Piezometric level line at top of layer
     99 - Piezometric level line at bottom of layer
      5 - Boundarynumber at top of layer
      4 - Boundarynumber at bottom of layer
    6 - Layer number, next line is material of layer
      Gorkum licht
     99 - Piezometric level line at top of layer
     99 - Piezometric level line at bottom of layer
```

```
       6 - Boundarynumber at top of layer
       5 - Boundarynumber at bottom of layer
     7 - Layer number, next line is material of layer
       Hollandveen
      99 - Piezometric level line at top of layer
      99 - Piezometric level line at bottom of layer
       7 - Boundarynumber at top of layer
       6 - Boundarynumber at bottom of layer
     8 - Layer number, next line is material of layer
       Gorkum zwaar 1
      99 - Piezometric level line at top of layer
      99 - Piezometric level line at bottom of layer
       8 - Boundarynumber at top of layer
       7 - Boundarynumber at bottom of layer
     9 - Layer number, next line is material of layer
       Hollandveen
      99 - Piezometric level line at top of layer
      99 - Piezometric level line at bottom of layer
       9 - Boundarynumber at top of layer
       8 - Boundarynumber at bottom of layer
    10 - Layer number, next line is material of layer
       Duinkerken Zand
       1 - Piezometric level line at top of layer
       1 - Piezometric level line at bottom of layer
      10 - Boundarynumber at top of layer
       9 - Boundarynumber at bottom of layer
    11 - Layer number, next line is material of layer
       Duinkerken Klei
       1 - Piezometric level line at top of layer
       1 - Piezometric level line at bottom of layer
      11 - Boundarynumber at top of layer
      10 - Boundarynumber at bottom of layer

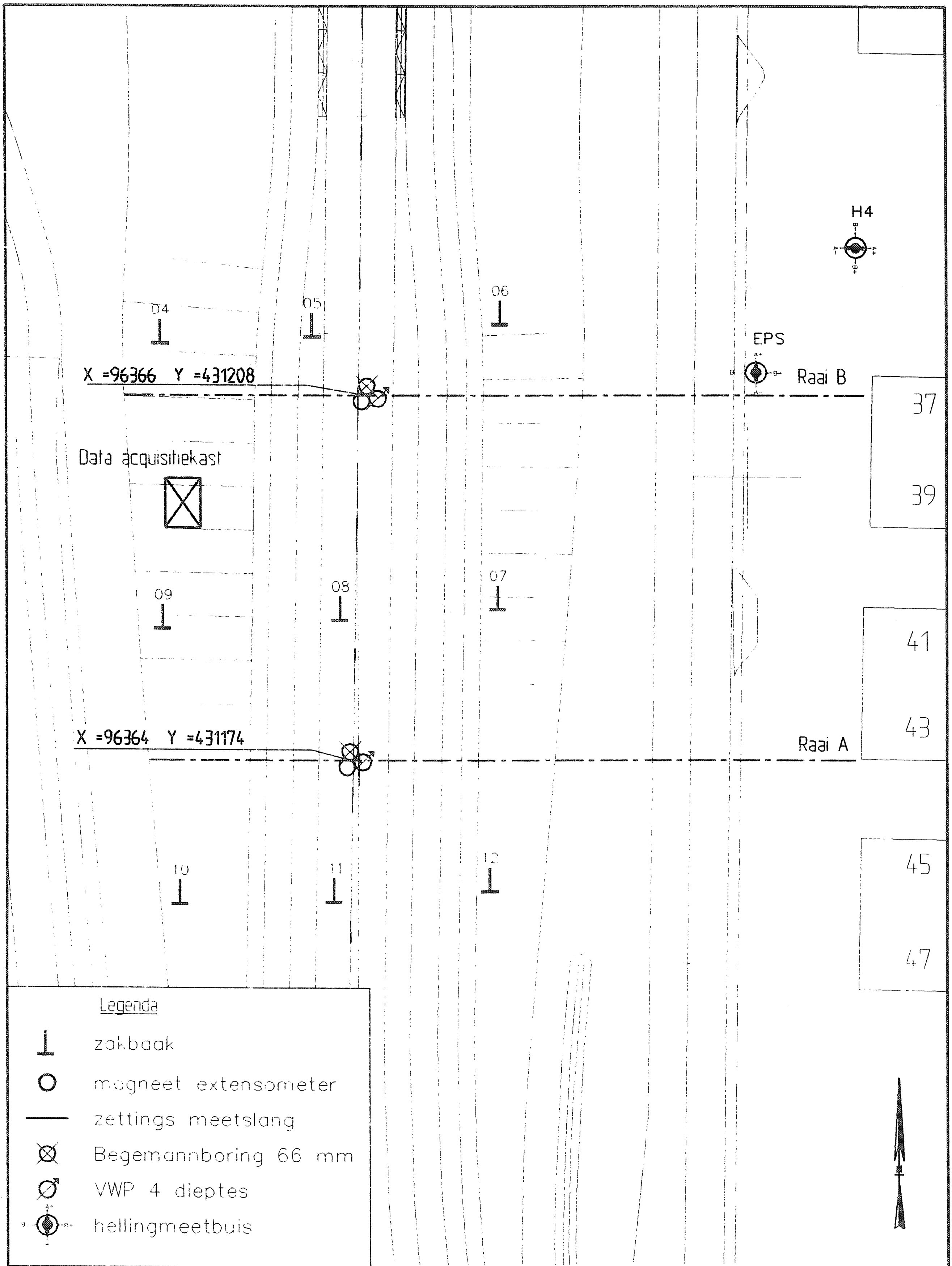
[END OF GEOMETRY DATA]
[RUN IDENTIFICATION]
Geometrie en ophoging Barendrechtse weg
Illustratie isotachenmodel
[MODEL]
1 : Dimension = 2D
0 : Calculation type = Darcy
2 : Model = Isotache
1 : Strain type = Natural
1 : Vertical drains = TRUE
0 : Fit for settlement plate = FALSE
[VERTICALS]
    300 = total Mesh
    1 = number of items
      17.000     0.000 = X, Z
[WATER]
        9.81 = Unit Weight of Water
     2000000 = Bulk Modulus of Water
[NON-UNIFORM LOADS]
   21 = number of items
ontgraving klei
           0       -19.20       -19.20 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       7.500       -0.500 = X, Y
       8.000       -0.800 = X, Y
      36.000       -0.800 = X, Y
      36.500       -0.500 = X, Y
spuitkade 1 west
           0        19.20        19.20 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       0.000       -0.500 = X, Y
       2.500        1.000 = X, Y
       5.000        1.000 = X, Y
       7.500       -0.500 = X, Y
spuitkade 1 oost
```

```
          0        19.20        19.20 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
      36.500       -0.500 = X, Y
      39.000        1.000 = X, Y
      41.500        1.000 = X, Y
      44.000       -0.500 = X, Y
ophoogslag 1 tot mv 20
         12        20.00        20.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
       7.500       -0.500 = X, Y
      36.500       -0.500 = X, Y
ophoogslag 1 zand 20
         12        20.00        20.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
       5.333        0.800 = X, Y
      38.668        0.800 = X, Y
ophoogslag 1 tot mv eraf
         26       -20.00       -20.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       5.333        0.800 = X, Y
       7.500       -0.500 = X, Y
      36.500       -0.500 = X, Y
      38.668        0.800 = X, Y
ophoogslag 1 zand eraf
         26       -20.00       -20.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       7.500       -0.500 = X, Y
       8.000       -0.800 = X, Y
      36.000       -0.800 = X, Y
      36.500       -0.500 = X, Y
ophoogslag 1 tot mv 18
         26        18.00        18.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
       7.500       -0.500 = X, Y
      36.500       -0.500 = X, Y
ophoogslag 1 zand 18
         26        18.00        18.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
       5.333        0.800 = X, Y
      38.668        0.800 = X, Y
spuitkade 2 west
         44        18.00        18.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       2.500        1.000 = X, Y
       5.000        2.600 = X, Y
       6.500        2.600 = X, Y
       9.500        0.800 = X, Y
spuitkade 2 oost
         44        18.00        18.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
      32.000        0.800 = X, Y
      34.500        2.800 = X, Y
      36.500        2.800 = X, Y
      39.000        1.000 = X, Y
ontgraving middendeel
         44       -18.00       -18.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
      11.000        0.800 = X, Y
      11.500        0.000 = X, Y
      29.000        0.000 = X, Y
      29.500        0.800 = X, Y
ophoogslag 2 zand 20a
        117        20.00        20.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
      11.000        0.800 = X, Y
      29.500        0.800 = X, Y
ophoogslag 2 zand 20b
        119        20.00        20.00 = Time, Gamma dry, Gamma wet
```

```
    2 = Number of co-ordinates
       6.667         2.500 = X, Y
      34.125         2.500 = X, Y
ophoogslag 2 zand erafa
        131        -20.00        -20.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       6.667         2.500 = X, Y
       9.500         0.800 = X, Y
      32.000         0.800 = X, Y
      34.125         2.500 = X, Y
ophoogslag 2 zand erafb
        131        -20.00        -20.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
      11.000         0.800 = X, Y
      11.500         0.000 = X, Y
      29.000         0.000 = X, Y
      29.500         0.800 = X, Y
ophoogslag 2 zand 18a
        131         18.00         18.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
      11.000         0.800 = X, Y
      29.500         0.800 = X, Y
ophoogslag 2 zand 18b
        131         18.00         18.00 = Time, Gamma dry, Gamma wet
    2 = Number of co-ordinates
       6.667         2.500 = X, Y
      34.125         2.500 = X, Y
ophoogslag 3 zand 18
        192         18.00         18.00 = Time, Gamma dry, Gamma wet
    4 = Number of co-ordinates
       5.000         2.600 = X, Y
       6.000         3.450 = X, Y
      38.000         3.450 = X, Y
      41.500         1.000 = X, Y
ophoogslag 4 zand 18
        332         18.00         18.00 = Time, Gamma dry, Gamma wet
    6 = Number of co-ordinates
       6.000         3.450 = X, Y
       6.500         3.800 = X, Y
      22.000         3.800 = X, Y
      24.500         4.500 = X, Y
      36.500         4.500 = X, Y
      38.000         3.450 = X, Y
ophoogslag 5 zand 18
        599         18.00         18.00 = Time, Gamma dry, Gamma wet
    6 = Number of co-ordinates
       6.500         3.800 = X, Y
      10.000         7.050 = X, Y
      22.000         7.050 = X, Y
      23.000         8.350 = X, Y
      32.000         8.350 = X, Y
      36.500         4.500 = X, Y
[WATER LOADS]
    0 = number of items
[OTHER LOADS]
    0 = number of items
[CALCULATION OPTIONS]
4 : Precon. pressure within a layer = Variable, correction at t=0 [days]
0 : Imaginary surface = FALSE
1 : Submerging = TRUE
0 : Maintain profile = FALSE
Superelevation
     0 = Time superelevation
    10.00 = Gamma dry superelevation
    10.00 = Gamma wet superelevation
1 : Dispersion conditions layer boundaries top = Drained
1 : Dispersion conditions layer boundaries bottom = Drained
0 : Stress distribution soil = Buisman
```

```
1 : Stress distribution loads = Simulate
 0.10 = Iteration stop criteria submerging [m]
 0.10 = Iteration stop criteria desired profile [m]
    1.00 = Load column width imaginary surface [m]
    1.00 = Load column width non-uniform loads [m]
    1.00 = Load column width trapeziform loads [m]
10000 = End of consolidation [days]
8 = Number of subtime steps
1.000E+00 = Reference time
[RESIDUAL TIMES]
0 : Number of items
[PORE PRESSURE METERS]
    0 = number of items
[NON-UNIFORM LOADS PORE PRESSURES]
   21 = number of items
ontgraving klei
       0.000 = Top of heightening
spuitkade 1 west
       0.000 = Top of heightening
spuitkade 1 oost
       0.000 = Top of heightening
ophoogslag 1 tot mv 20
       0.000 = Top of heightening
ophoogslag 1 zand 20
       0.000 = Top of heightening
ophoogslag 1 tot mv eraf
       0.000 = Top of heightening
ophoogslag 1 zand eraf
       0.000 = Top of heightening
ophoogslag 1 tot mv 18
       0.000 = Top of heightening
ophoogslag 1 zand 18
       0.000 = Top of heightening
spuitkade 2 west
       0.000 = Top of heightening
spuitkade 2 oost
       0.000 = Top of heightening
ontgraving middendeel
       0.000 = Top of heightening
ophoogslag 2 zand 20a
       0.000 = Top of heightening
ophoogslag 2 zand 20b
       0.000 = Top of heightening
ophoogslag 2 zand erafa
       0.000 = Top of heightening
ophoogslag 2 zand erafb
       0.000 = Top of heightening
ophoogslag 2 zand 18a
       0.000 = Top of heightening
ophoogslag 2 zand 18b
       0.000 = Top of heightening
ophoogslag 3 zand 18
       0.000 = Top of heightening
ophoogslag 4 zand 18
       0.000 = Top of heightening
ophoogslag 5 zand 18
       0.000 = Top of heightening
[OTHER LOADS PORE PRESSURES]
    0 = number of items
[CALCULATION OPTIONS PORE PRESSURES]
1 : Shear stress = TRUE
1 : calculation method of lateral stress ratio (k0) = Nu
[VERTICAL DRAIN]
0 : Flow type = Radial
     -10.000 = Bottom position
       0.000 = Position of the drain pipe
       1.000 = Centre to centre distance
       0.070 = Diameter
```

```
1 = number of items
     0.000     0.00      -0.700       0.00 = Time, Air pressure, Water level, Tube
pressure
[1D GEOMETRY]
        0.000 = Phreatic level
        0.000 = Bottom level
            0 = Number of layers
[END OF INPUT FILE]
```



0 5 10 15 20 25m
scale 1:500

Filenaam	B-SI-A1 .dwg
Afdeling	300
Gewijzigd	2002-12-02

	datum 2002-12-02	get. WTH
BARENDRECHTSEWEG Section lines A and B	CO-710402	gez.
SITUATION	ANNEX A1	form. A4